CONSOLIDATION BEHAVIOR OF EXPANSIVE SOIL: THE CASE OF BOLE BESHALE IN ADDIS ABABA

By

TESFALEM G/MARIAM

A THESIS SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING FOR PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN GEOTECHNICAL ENGINEERING

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

JUNE 2018

APPROVAL PAGE

This M.Sc Thesis entitled with "**CONSOLIDATION BEHAVIOR OF EXPANSIVE SOIL: THE CASE OF BOLE BESHALE IN ADDIS ABABA**" has been approved by the following examiners in partial fulfillment of the requirement for the degree of Master of Science in Geotechnical Engineering.

Date of Defense: January 28, 2018

Principal Advisor

1. Dr. Trufat Hailemariam (Advisor)	________________ Signature	____________ Date

Members of the Examining board:

1. Dr. Aregaw Asha Principal Examiner	________________ Signature	____________ Date
2. Dr. Melaku Sisay Co-Examiner	________________ Signature	____________ Date
3. Dr. Melaku Sisay ERA PG, Programme Coordinator	________________ Signature	____________ Date
4. Mr. Simon G/Egiziabher Head, Civil Eng'g Department	________________ Signature	____________ Date
5. Dr. Brook Abate Dean, College of Architecture And Civil Engineering	________________ Signature	____________ Date

Acknowledgements

This is a great opportunity for me to express my deepest appreciation to Dr. Trufat Hailemariam, Assistant Professor, School of Earth Sciences, Addis Ababa University, for his advice, guidance and support were invaluable throughout my graduate studies and in the completion of this research. Without his expertise, understanding, meticulous comments, and generous guidance this Thesis would not be possible.

I'm highly indebted and thoroughly grateful to Dejenie Abere from Ministry of water, Irrigation and electricity for his immense interest on my research, for providing me with materials, for inspiring and encouraging me upon difficulties, for criticizing and improving my technical writing skills, and for being constant sources of motivation.

I would also like to thank all my office colleagues and other friends, together with whom I have shared both my happiness and sorrows over the last four years as a postgraduate student.

I want to express my acknowledgments to my family, who always provide their love, support, and encouragement. Without their support, this dissertation would not be possible.

Table of Contents

List of Tables

List of Figures

Abbreviations

PVC	Potential Volume Change
CEC	Cation Exchange Capacity
AASHTO	American Association of State Highway Transport Officials
NMSA	National Meteorological Services Agency
FS	Free Swell
IL	Incremental Loading
USCS	Unified Soil Classification System
LL	Liquid Limit
PI	Plastic Index
PL	Plastic Limit
e_0	Initial Void Ratio
Gs	Specific Gravity
Av	Coefficient of Compressibility
Cc	Compression Index
Cv	Coefficient of Consolidation
Mv	Coefficient of volume Compressibility

Abstract

This study is carried out Bole Beshale in Addis Ababa with the aim to assess Consolidation Behavior of Expansive Soil. Expansive nature of soils often controls the design and cost of construction projects, predominantly light weighted and shallowly founded structures.

A geotechnical secondary data, a large quantity of drilling and laboratory testing data, from ongoing projects were taken and filtered out and utilized for this research. From Bole Beshale 40/60 condominium site, out of 116 boreholes a total of 38 borehole (which serves to build 19 blocks) were purposefully selected to represent the study area.

Relevant laboratory test results in the selected boreholes are also considered and data obtained from the field investigation were systematically organized to correlate in the selected section. Index properties shows that the soils are classified as high plastic silt (MH) and high plastic clay (CH) in Unified Soil Classification System (USCS). These soils show high plasticity index to imply that the soil contains dominantly Smectites clay minerals. The soil exhibited medium to high compressibility and also exhibit high rates of consolidation.

As liquid limit (LL) and plasticity index (PI) increases, compression index (Cc) increases. Correlations are established using LL, PI, and Cc. And shows that when liquid limit (LL) and plasticity index (PI) increases, compression index (Cc) increases. Correlation between both calculated and laboratory measured compression indices were found to be strong.

Keywords*: Expansive soils, liquid limit, Plasticity index, consolidation, compressibility*

CHAPTER ONE

INTRODUCTION

1.1. Back ground of the study area

Ethiopia is a rapid growing country that has been scoring a double digit economic growth for a decade. Infrastructural facilities that have been put in place for decades and a half namely, roads, schools, Mega projects, health centers and residential buildings are part of the growth. Residential buildings are also getting the major portion of the infrastructures development.

Many scholars mentioned that identifying and evaluating the sub surface soil profile by detail site investigation and play a key role for safe and economical design of engineering structure.

Soil is one of the most important engineering materials. It is involved in most civil works and affects their success. Soil is an essential component in civil engineering infrastructure development. It often controls the design and cost of construction projects, predominantly light weight and shallow founded structures. Some soils, such as expansive soils, are known to be problematic and require particular attention (Jones and Holtz, 1973).

Soils are composed of a variety of material, most of which do not expand in the presence of moisture and some of which will expand with change of temperature (Holtz & Kovacs, 1981). When a soil contains a large amount of expansive minerals, it has the potential of significant expansion. Even though expansive soil cause enormous amount of damage, most people have never heard of them. (Kemp et al., 2005).

Expansive soil pose problem to civil engineers in general and to geotechnical engineers in particular (Chen 1988). They cause damage to structure founded in them because of their potential to react to changes in moisture regime. They undergo severe volume changes corresponding to change in moisture content. They swell or increase when they imbibe water and shrinkage, they result in harmful cracking of lightly loaded civil engineering structures such as foundations, retaining walls, pavements, airports, sidewalks, canal beds and lining (Chen 1998). The amount of volume change by an expansive soil depends greatly on its mineralogical composition. However, a number of clay minerals are expansive. These include: smectite, bentonite, montmorillonite, beidellite, vermiculite, attapulgite, nontronite, and chlorite show extreme variation up on wetting and drying (Omari and Hamodi, 1991) and also some physical factor such as initial water content, initial density, amount and type of compaction also influence the potential of expansive soils (Baser, 2009).

Each year, Engineering problems due to expansive soil have been reported in many countries all around the world. They cause millions of dollars due to their sever damages to house, other buildings, roads, pipelines, and other structure. This is more than twice the damage from floods, hurricanes, tornadoes, and earthquakes combine (nelson and miller, 1992). The damages are most common especially in the arid and semi-arid regions of the world (Mokhtari, M & M. Dehghani, 2012).

According to (Ashenafi Tamerat, 2013) Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas are partly covered by expansive soil. According to (Lucian, 2011), defined expansive soil as a potential natural hazard, which are problematic to engineering structure and which can cause extensive damage to structure because of their tendency to swell during wet season and shrink during dry season if not adequately treated.

This study was conducted in apartment building project which is located in Addis Ababa, around Bole Beshale site, Lot-03 with the aim to assess the consolidation behavior of expansive soil on foundation and proposed basic recommendations for those buildings which are constructed in the study area.

1.2. Description of the Research Problem

According to the studies, currently, more than three million people reside in the city of Ethiopia and due to rural-urban migration the population growth is increasing and there is a clear gap between house supply and demand. In order to fill the gap, the government prepare the house construction task through various schemes. The most known are condominium house construction.

One of those project are apartment building project which is located in Addis Ababa, around Bole Beshale site, Lot-3.The proposed development consist of the construction of eight, ten, fourteen stories building with basement and ground floor (B+G+8), (B+G+10), (2B+G+14).The project site is generally characterized by nearly flat to sloping topographic feature and it is erratic, it varies laterally and vertically in terms of occurrence of soil and rock layers.

Laboratory test results show that the top and the underling soil layers are highly expansive and have a very high potential for volume change. However, proper settlement assessment is rarely considered pertaining to building loads. In the present study, an attempt is made to carry out a settlement characterization on these soils for building foundations. All relevant factors affecting soil behavior and other ground conditions are considered so that a clear understanding on engineering behavior of expansive soils could be assessed.

1.3. Objectives

1.3.1 General Objective

The general objective of this study is to assess Consolidation Behavior of Expansive Soil in Addis Ababa taking a case study in Bole Beshale area

1.3.2 Specific Objectives

To meet the general objective, the following specific objectives were designed:

- ✓ To define and characterize expansive soils in Bole Beshale area.
- ✓ To assess index and engineering properties of the expansive soils.

1.4. Methodology

To achieve the objectives of the study, the process started from literature review, field observations, and collection of samples from analysis of laboratory test results.

The following systematic methodology has been adopted:

- ✓ The literature survey encompasses both published and unpublished reports of investigations, textbooks and journals that were obtained from different sources and previous researches which were related to the topic.
- ✓ Data and generation on drilling, in-situ testing, logging, sampling, and laboratory testing were taken from secondary sources (from ongoing projects of study area) based on the scope that has propose. In bole beshale 40/60 condominium site we find 116 boreholes. From this Bore hole, 284 soil samples were tested in the laboratory. Currently; 58 blocks are being constructed by Addis Ababa housing Agency. However; for this study 38 borehole (which serves to build 19 blocks) were purposefully selected from the representative transect lines. From both transects line 1-1 and 2-2 were selected 14 boreholes each. And on transect 3-3, 10 boreholes were selected as shown in figure 1.1. The reason transect line 3-3 borehole number is smaller because of playground opening. The criterion for selection of transects lines on the bases of topography from bottom to top to consider dominance of expansive soil.

Figure 1.1: Transects line in Bole Beshale (study area)

- ✓ Organization of data and generating a geotechnical database.
- ✓ Geotechnical secondary data was analyzed and processed by means of computer software (Microsoft Excel) and derived relevant correlations

1.5. Significance of the Research

Expansive soil constitute a significant hazard to engineering construction especially for light weight structure in terms of their ability to swell or shrink, usually caused by seasonal change in moisture content (Cheny, 1986). Accordingly, this research work may be considered as very important from the point of view of analysis of soil to identify soil type and determine their expansive property on building foundation.

The results and findings are expected to:

- Predict the compressibility of expansive soil based on index properties in solving the Site problem in a quick and efficient way.
- Provide valuable information for land use planning, particularly for buildings, and other Civil engineering construction activities; in addition, may be applied to similar area where expansive soils are present.
- May contribute some input to the ongoing nation-wide studies on expansive soils.

1.6 Organization of the research

The present study was conducted by compiling geotechnical data from historical borehole log reports and laboratory test results, engineering geological maps, ongoing projects and visual observations in study area. To present the results of the research in a systematic manner, it is divided into six chapters and the scheme of presentation is as follows:

Chapter-One comprises the introduction, which includes the background of the study area, description of the research problem, objectives, methodology and significance of the research.

Chapter –Two presents the literature review that comprises a brief description of the theoretical basis and previous works relevant to the present research.

Chapter-Three presents a description of the study area including geographical location, topography and drainage, regional and site geology, climate, hydrogeology, soil, and Expansive soil in the study area.

Chapter-Four presents the characterization of the expansive soil that describes distribution and genesis, soil distribution in the study area, index property, soil classification and engineering properties.

Chapter -Five presents consolidation, factors affecting the consolidation characteristics of clay soils, testing procedures, compressibility characteristics, results of consolidation tests and consolidation characteristics.

Chapter-Six presents the overall conclusions and recommendations that are forwarded through the present research work

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Soil is a mixture of various sizes of particle like gravel, sand, silt and clay. Gravel and sand are the coarse fractions and they are considered inert materials because of their insignificant activity. In contrast, clay and silty clay are particles of ultra-fine size in the form of platelets. They carry an unbalanced negative electric charge on their surface. This electric charge and large specific surface they possess render them highly active. They can absorb water as well as the positively -charged ions from the salts in water to neutralize the electric charge they carry on their surface (Kulbir Singh, 1995). The amount of water absorbed depends on the type of the clay mineral presenting the soil. Three most common minerals present in clay are Kaolionite, Illite and Montmorillonite and their capacity to adsorb water increases in that order, therefore, the greater the percentage of Montmorillonite mineral present, the greater would be the expansive nature of the soil (Omari and Hamodi, 1991).

2.2 What are expansive soils?

Essentially expansive soil is one that changes in volume in relation to change in water content. Here the focus is on soils that exhibit significant swell potential and in addition shrinkage potential also exists (Chen1988, Nelson and Miller, 1992). The behavior of expansive soils varies from place to place depending upon the type of parent material, climate and topography (Morin, 1971).

Expansive soil can be classified in to two groups with respect to the parent materials. the first group comprises the basic igneous rocks, such as the basalts of Deccan plateau in India, the Dolerite sills and dykes in the central region of south Africa and Gabbros and norities of west of Pretoria north, Transvaal. In these soils the Feldspar and Pyroxene minerals of the parent rocks have decomposed to form Montmorillonite and other secondary materials. The second group comprises the sedimentary rocks that contain

Montmorillonite as a constituent which breaks down physically to form expansive soils.in North America, bedrock shale found in the Pierre formation and more recent Larame and Denver formation are example of this type of rock in Israel, there are the marls and lime stone and in south Africa, the shale of the Ecca Series (Gilloth, 1962; Grim, 1953).

2.3 Properties of Expansive Soils

Excluding deep underground excavation (e.g. tunnels), shrinkage and swelling effects are restricted to the near-surface zone; significant activity usually occurs to about 3m depth, but this can vary depending on climate conditions. the shrink-swell potential of expansive soils is determined by its initial water content; void ratio; internal structure and vertical stresses, as well as the type and amount of clay minerals in the soil(Bell & Culshaw,2001).These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite,illite and chlorite. Generally, the larger the amount of these minerals present in the soil, the grater the expansive potential. However, these expansive effects may become "diluted" by the presence of other non-swelling minerals such as quartz and carbonate (Kemp et al., 2005).

The key aspects of expansive soil behavior, however, are a soil exposure of water induced volume change. When soils with a high expansive potential are present they will usually not cause a problem as long as their water content remains relatively constant. This largely control by (Houstonet al., 2011).Soil properties, e.g. mineralogy and water content variations both temporally and spatially

2.3.1 Mineralogical aspects of expansive soils

Expansive soils are composed of hydrophilic clay mineral. Clay particle are very small and their shape is determined by the arrangement of the thin crystal lattice layers that they form, with many other elements which can become incorporated into the clay mineral structure. (Mitchell & Soga, 2005).

The shrink-swell potential of expansive soils is determined by its initial water content; void ratio; internal structure and vertical stress, as well as the type and amount of clay mineral in the soil (Bell & Culshaw, 2001).

The two basic structural units of the lattice structure of clay soil first comprises two-layered sheet made from units of six hydroxyl (or oxygen) ions in octahedral coordination with aluminum, iron or magnesium ions and the second comprises sheet silicate structure(Figure 2.1). The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions. The aluminum octahedron consists of an aluminum atom surrounded octahedrally by six oxygen ions. There are valence imbalances in both units, resulting in net negative charges. The basic units, therefore, do not exist in isolation but combine to form sheet structures. When each oxygen atom is shared by two tetrahedral, a plate-shaped layer is formed. Similarly, when each aluminum atom is shared by two octahedrons, a sheet is formed. The silica sheets and the aluminum sheets combine to form the basic structural units of the clay particles. Various clay minerals differ in the stacking configuration (Murray, 2007; Craig, 2004).

2.3.2 Seasonal variation in water content

The critical climate type for expansive clays is one in which there is a distinct wet period followed by a hot dry period. During the wet period the soil imbibes water and swelling occurs, while during the subsequent dry period, the soil loses moisture via evaporation, and volume reduction occurs. The severity of the weather varies, however, from year to year. Moisture content in clay sub-grades under pavements or foundations is of almost importance as they determine the shear strength of the clay and its bearing capacity .If the clay is of the swelling type, these moisture changes also act to deform the pavement or foundations and thus may affect its usefulness. The moisture content in a soil mass varies not only due to artificial wetting, but also evaporation, change in thermal conditions, and transpiration by vegetation, precipitation etc. (Hussein Elarabi,2014).

In addition to these factors the annul fluctuation in moisture content in the upper layer of the soil depend upon evaporation and wetting due to atmospheric precipitation, which is related to the precipitation balance. The soil surface is subjected to diurnal, annual and perennial temperature fluctuation. The temperature varies not only at the surface, but also vertically along the depth of the soil mass where it fluctuates (Nelson and Miller, 1992) Thus, in a layer at the surface of the soil the moisture content is observed to be smaller compared to that in layers which are located at depth. The moisture content of the upper layer does not remain constant but varies with time. This leads to violation of equilibrium conditions which results in the emergence of various forces which cause migration of moisture (Nelson et al., 2010). In certain cases this affect the task of building construction since an increase and decrease in the moisture content of expansive soil leads to an increase or decrease in their volumes which may cause damage to the structure.

2.4 Distribution of Expansive Soil in Ethiopia

Expansive soil is found through many regions of the world, particularly in arid and semi-arid regions, as well as where wet conditions occur after prolonged period of drought. Their distribution is dependent on geology (parent material), climate, hydrology, geomorphology and vegetation (Mokhtari, M &M. Dehghani, 2012). Expansive soil is known to be spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Abeba areas and central part of Ethiopia following the major trunk roads like Addis –ambo, Addis-Woliso, Addis-Debreberhan, Addis-Gohatsion, Addis-Modjo are some of the area covered by expansive soils. Areas like some part of Mekele, Gondor, Bahirdar, Deberberihan and Gambela are also known to be partly covered by expansive soils (Tibebu Solomon, 2015).

2.5 Identification of Expansive Soils

The key to all expansive soil classification system is the method of measuring swell potential either in the field or in laboratory, since soils are rated by their measured swell potential.

2.5.1 Field investigation and Identification

It is evident that expansive soil deposit can be recognized in the field through visual inspection. The method is simple and easy to use.

Some of the important field identification method that indicates the potential for expansive soil is polished with a smooth object such as the top of a finger nail.

- The wet sample of the soil is sticky and it will be relatively difficult to clean the soil from the hands.
- The appearance of cracking in nearby structure.
- They usually have a color of black and gray.
- In the region where there is seasonal moisture variation
    - Open or closed fissures,(a joint or similar discontinuity)
    - Slickenside,(highly polished or glossy fissure surface)
    - Shattering or micro-shattering, (presence of fissure forming granular fragment of clayey soils).

2.5.2 Experimental Identification

Generally, there are different methods of identifying expansive soil in the laboratory and field experiment. These include mineralogical, Direct and indirect identification techniques (Chen, 1988).

Type of clay mineral is a fundamental factor, which determines the expansive behavior of a soil. Mineralogical test is used to identify this mineral. This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and charge deficiency and surface activity of clay particle. These properties are a fundamental factor controlling expansive soil behavior. The various techniques under this method are X-ray diffraction, Differential thermal analysis, Dye absorption, Electron microscope Base Exchange capacity, Infrared spectroscopy and Radio frequency electrical dispersion. But these methods are not suitable for routine test because, they are time consuming, require expansive test equipment and, the results are interpreted by specially trained technicians.

Indirect methods are used to investigate the swelling potential of a soil by examining other parameter, which indirectly yield excellent indices of expansive properties. Such tests are easy and can be performed in average soil mechanics laboratory. The commonly used test here is the index property tests (consist of Grain Size analysis, liquid limit, plastic limit, shrinkage limit, free swell and vertical swell).Also Cation exchange capacity(CEC) and potential volume change (PVC)test can be used.

2.6 Classification Methods of Expansive Soils

The different classification systems are categorized into two:-

- ✓ General classification system:- which have evolved over many years and are based largely on correlation with actual performance
- ✓ Classification specific to expansive soil:-these systems are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

The most widely used general classification systems are: - The Unified Soil Classification System (USCS) and the American Association of State Highway Transport Officials (AASHTO) method make use of the liquid limit and plasticity index of the soil (Das, 2011).The Classification system developed based on shrinkage limit and linear shrinkage (Altmeyer, 1956), index property (Kanntey and Brink, 1952) to give qualitative rating on the expansiveness of the soil.

2.7 Mechanics of Swelling

Swelling of expansive soils will take place under change in the environment of the soil. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. There must be a potential gradient, which can cause water migration and a continuous passage through which water transfer can take place (Chen, 1988). The potential gradient in expansive soils can be due to seasonal moisture fluctuation or thermal gradient, which can cause vapor and liquid moisture transfer (Bell, 1999). Vapor transfer plays an important role in providing the means for the volume increase of expansive soils (Ross, 1978).

2.7.1 Factors Influencing Swelling of Expansive Soil

Swelling and shrinkage potential of expansive soils depend on several factors, such as type and amount of soil constituent clay minerals; initial moisture content, and the soil water chemistry contained within voids; initial density or void ratio; grain size, structure and soil fabric; the state of stress, which is the applied stress level and stress history (Bell, 1999; Chen, 1988; Nelson and Miller, 1992).

2.8 Previous Works on Expansive Soils

Researches on the engineering properties of expansive soils in different parts of the world have been done. Some of the researches undertaken are; Pavements on expansive soils are comprehensively summarized by Holland and Richards (1982). The paper briefly outlines the worldwide problem that result from the construction of pavements on expansive soils. Basic concepts of clay soils heave, including the effects of site drainage and loading were discussed. A brief presentation of the major techniques most commonly used in an attempt to overcome the problems of expansive soils was also included in the paper.

Netterberg (2001) gave descriptions of a full-scale road experiment on the Pretoria Warm bath freeway in South Africa employing four different counter measures against damage due to the highly active clay road bed. The entire four counter measures used, i.e. 1.0m replacement, pre-wetting (by rainfall) alone, pre-wetting and membranes; and a combination of 0.5m replacement, pre-wetting and membranes which have performed satisfactorily.

The Geological Society (1997) reviewed the nature of different tropical residual soils and gives valuable information on how their peculiarities influence their testing and use in civil engineering. Each type of residual soil has well defined characteristics derived from the nature of the parent rock and the climate prevailing in recent geological times. Thus, whenever similar climatic and geological conditions occur, the same types of residual soils are evolved, sometimes at different stages of development. The report discuss the origin, weathering processes and distribution of tropical residual soils and gives a general guidance in predicting the soil groups most likely to be found under any particular combination of parent materials and environment.

Morin and Parry (1971) presented the occurrences and properties of the black and red clay soils. They pointed out that the black clay soils are difficult to be utilize in construction because they contain a large percentage of expansive clay minerals.

2.9 Consolidation

2.9.1 Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. The soil layers are unable to spread laterally as the surrounding soil strata confines them. Hence there must be adjustment to the new pressure by vertical deformation. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. The vertical compression of the soil mass under increased pressures is thus made up of the following components:

- ✓ Deformation of the soil grains
- ✓ Compression of water and air with in the voids
- ✓ An escape of water and air from the voids

It is quite reasonable and rational to assume that the solid matter and the pore water relatively are incompressible under the loads encountered. The change in volume of the soil mass under imposed stresses must be only due to the escape of water and air. Generally, the volume change in a soil deposit can be divided in to three stages (Arora, 2004).

a) Initial consolidation:

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles.

b) Primary consolidation:

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as excess pore water, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the permeability of the soil and is, therefore, time dependent. The reduction in volume is called primary consolidation. In fine grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability. As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles.

c) Secondary consolidation

The reduction in volume continues at a very slow rate even after the excess pore water pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in the volume is the called secondary consolidation. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the adsorbed water to the new stress system. In most inorganic soil, it is generally small.

CHAPTER THREE

DESCRIPTION OF THE STUDY AREA

3.1 General

The formation of the different types of soils depends on the prevailing environmental factors of an area. The climatic conditions, the geologic and physiographic setup of an area have an impact on the formation of expansive soils since these soils need specific condition to be fulfilled. Climatic influence is more significant in Ethiopian soil development than petrology of the parent rock (Morin and Parry, 1971). Therefore, due to the fact that the engineering properties of expansive soil of Ethiopia are different from the same soil in other locality accordingly, a considerable part of central and eastern Addis Ababa is occupied by expansive soils which are problematic when used as foundation materials (Alemayehu Tefera and Solomon Yohannes, 1986).

This chapter deals with the description of geologic, topographic, climate and hydrogeologic conditions which influenced the formation of expansive soils in the study area. This section also includes description of the seismic condition and soil type distribution in the study area and its surrounding.

3.2 Location

The study area is located in the Bole sub-city of Addis Ababa, capital city of Ethiopia. It is located 37P zone between 473983E, 489545E and 987384N; 997204N geographic coordinates in UTM and is situated at the western margin of the Main Ethiopian Rift with an aerial coverage of about 40km^2 in the Eastern part of Addis Ababa city. Location map of the study area is presented in Figure 3.1.

3.3 Topography and Drainage

Topography controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering. In addition to this, it also controls the effective age of the profile by controlling the rate of erosion of weathered material from the surface. Thus, deeper residual profiles will generally be found in valleys and gentle slopes rather than on high ground or steep slopes (Blight, 1997).

Bole sub city district is characterized by a highly flat relief with small hills. The general slope is about 0.9%, dipping from the northern or upper catchment part towards kality catchment. The highest elevation in bole behale is 2368.60 meters, while the lowest point in the study area is 2332.20meters (DEM).the steeper areas are the south edge and the western part, which consist mainly on small valleys bordering in the urbanized area.

The streams of Addis Ababa drain towards south from Entoto ridge, south east from Mountain Wecheca and Mountain. Furi and towards south west from Mountain Yerer and other elevated areas of the eastern outskirts of the city. The potential streams in the city are Little Akaki, Bantyiketu, Kurtume, Kebena, Ginifile, and Big Akaki. Other streams are intermittent in nature. Stream are dense with deep valleys on top of mountains such as Entoto ridge forming radial and dendritic drainage patterns. The drainage pattern is governed by the geology and physiographic set up of the area (Tamiru Alemayehu et al., 2006).The topography, accompanied with the drainage condition of the study area creates a suitable environment contributing to the development of expansive soils. In low lying area (Study area) where surface drainage is poor or there is no drainage in the study area and often water logged, dark colored (dark grey) soils are dominant. In general, it seems like the topography and drainage of the study area controls the type and distribution of soils.

3.4 Regional and Site Geology

3.4.1 Regional Geology

Addis Ababa city is situated in the western margin of the Main Ethiopian Rift and represents a transition zone between the Ethiopian Plateau and the rift with poorly defined escarpment.

The geology of Addis Ababa area is represented by four volcanic units dominated in the lower part by basaltic lava flows (Addis Ababa basalt), followed by a pyroclastic sequence, mainly formed by ignimbrites (Addis Ababa Ignimbrite), followed by central composite volcanoes (Central Volcanoes unit), and finally small spatter cones and lava flows (Akaki unit).

Addis Ababa basalt extensively crops out along Akaki, Kebena, and Dukem rivers at the east to southeastern part of Addis Ababa, and represents the oldest unit of the area. It consists of essentially sub-horizontal lava flows with thickness ranging from few meters up to 20m. Maximum exposed thickness was found east of Addis Ababa, along the Kebena River. Addis Ababa basalt is predominantly constituted by alkaline and olivine basalts with three main textural attributes, that is, porphyritic, aphyric, and sub-aphyric.

Addis Ababa Ignimbrite is exposed close to Addis Ababa along the Akaki and Kebena rivers. It overlies the Addis Ababa basalt and locally covers the products of the composite central volcanoes of Wechecha and Furi. The sequence is constituted by different flow units, consisting of pale-green to pale-yellow welded and crystal rich ignimbrites.

Central volcanoes unit includes the Yerer volcano and the product of the two composite volcanoes wechecha and Furi west and southeast of Addis Ababa, respectively. Wechecha and Furi volcanoes are two large edifices composed by predominant trachyte with minor pyroclastics. Yerer represents the largest volcanic edifice in the region, with a relief of 1000m from the plain and 14km wide along east-west direction. Products mainly consist of trachytes, even if pyroclastics are widespread mainly in the central part eastern sector. The highest part of Yerer volcano was affected by a more recent volcanic activity that produces spatter cones and associated basalt.

Akaki unit crops out east of Addis Ababa and consists of scoria and spatter cones with associated tabular lava flows and phreato- magmatic deposits. Alluvial deposits covering these units consists of regolith, reddish brown soils, talus and alluvium with maximum thickness of about two meters.

3.4.2 Site Geology

The geology of the project site is generally represented by different soil and rock layers up to the maximum drilled depth, 20.0m. The following section presents the site geology of respective parcels.

The geology of the project site is generally represented by different soil and rock layers up to the maximum drilled depth, 20.0m.

The top part of the project site around this area is dominantly covered with medium stiff to stiff, dark grey, high plastic, silty CLAY/clayey SILT (black cotton soil). Stiff to hard, light brown to light grey, high plastic, clayey SILT soil is observed immediately below the upper layer in some of the drilled boreholes. Following the upper soil layers, there is very stiff to hard, low to high plastic, silty SAND/sandy SILT soil (weathering product of IGNIMBRITE). Moderately strong, light grey, slightly weathered IGNIMBRITE rock layer is found to bottom depth in few boreholes.

3.5 Climate

Climate is the principal factor governing the rate and type of the soil formation. The two most important components of climate are precipitation and temperature. Ethiopia is in the tropical zone lying between the equator and the tropic of cnacrer.it has five different climate zones according to elevation. Kolla (tropical zone) is below 1830 meters in elevation. Woina dega (subtropical zone) includes the highlands areas of 1830-2440 meters in elevation. Dega (cool zone) is above 2440 meters in elevation. "Kur" (Alpine), above 3000m mean sea level and "Bereha" (Desert), less than 800m.

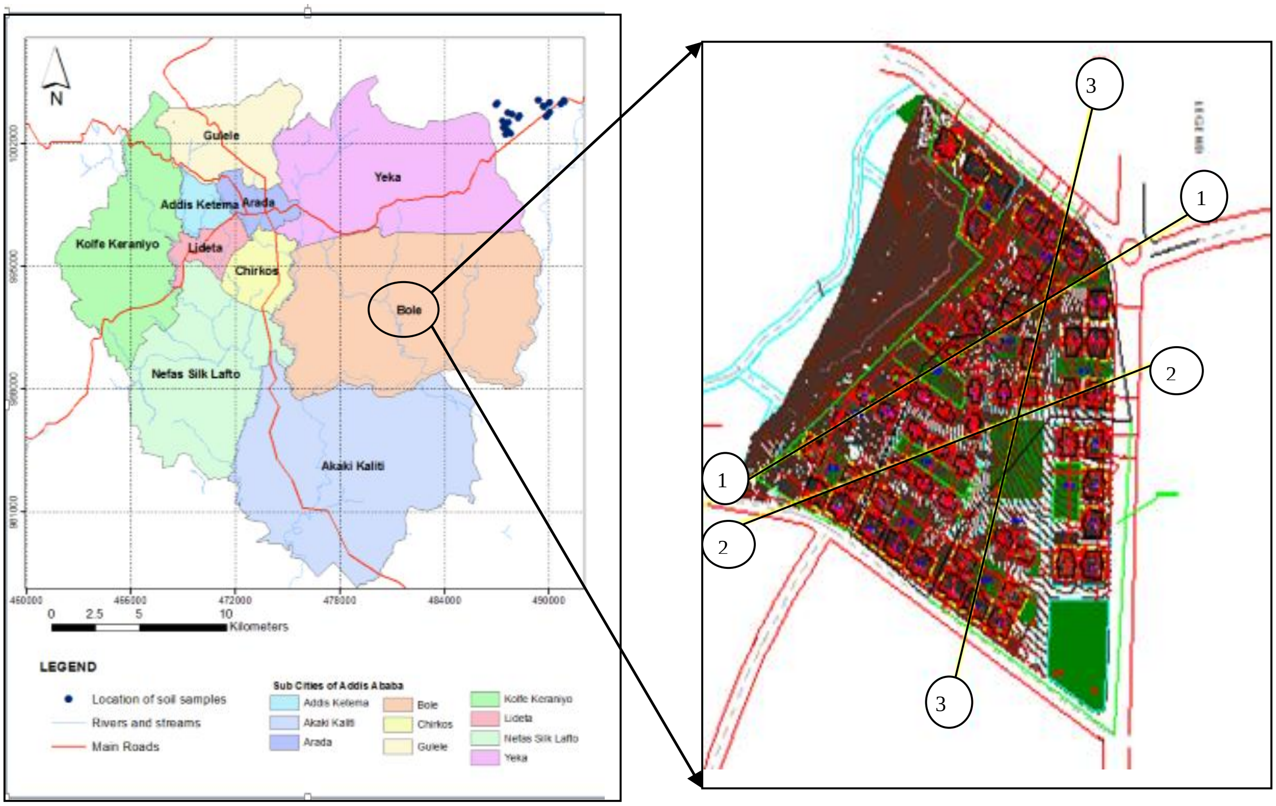

Figure 3.1: location map of the study area

Climatic patterns tend to produce different soil types depending on the degree of weathering on parental rocks responsible for the formation of flat topography in the study area. Bole sub city generally have a sub-tropical climatic condition.

3.5.1 Rainfall

Addis Abeba has three distinct seasonal periods with seven main rainy months from March to September. The dry season (Bega) is between the months of October to February, the short rain season is between March to May and the big rainy season is between the months of June to September (Feven Solomon, 2007).In Addis Ababa rainfall intensity variation is attributed to the differences in topography. The high elevated areas such as the Entoto receive relatively greater precipitation than lowland areas around the study area.

According to the data obtained from National Meteorological Services Agency (NMSA), the annual mean rainfall of Addis Ababa Bole station, Addis Ababa Observatory Tekelehaimanot station and Akaki Beseka) stations are 1091.3mm, 1205.2mm and 1154.7mm, respectively. Thus, the city receives mean annual rainfall of about 1150.4mm as shown in table 3.1. All stations are located at an elevation of 2350m, 2408m and 2000m a.s.l. respectively.

Table 3.1 Mean Monthly Rainfall (mm) in three stations of Addis Ababa (1971 to 2005)

Station	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Ann total
Akaki Mission	13.7	43.1	60.0	95.1	66.5	128.8	271.1	303.8	140.9	23.9	4.3	3.4	1154.7
AA observt	17.2	43.2	65.0	93.7	86.4	128.9	257.9	279.7	176.5	38.9	8.4	8.8	1205.2
Bole station	15.4	40.1	68.7	92.8	79.5	116.6	234.8	244.1	150.3	34.3	10.1	4.5	1091.3
Mean Monthly	15.43	42.1	64.6	93.9	77.5	124.8	254.6	275.9	155.9	32.4	7.6	5.6	1150.4

From 34 years data the heaviest amount of rain fall occur in the month of August, While the minimum amount of rainfall occur in December at Addis Abeba and Akaki mission stations and in November at Addis Abeba observatory (Tamiru Alemayehu et.al.2005). Furthermore, Addis Ababa Observatory which is located at a higher elevation than Addis Ababa Bole and Akaki Beseka stations, records greater amounts of mean annual rainfall (1205.2mm). This shows that there is a variation in the amount of rainfall within Addis Ababa with differences in altitude.

3.5.2 Temperature

Under normal conditions, air temperature decrease with increasing altitude at a mean rate of 0.7ºC for every 328 feet (Fetter, 1994 as cited in Hana Tibebu, 2008**).** This works also in Ethiopia where temperature decreases with increasing elevations. The average temperatures are typically tropical to sub-tropical and fluctuate by 50C between the coldest and warmest months (Griffiths, 1972 as cited in Habtamu Solomon, 2011). The mean monthly maximum and minimum temperature records of National Meteorological Services Agency (NMSA) stations in Addis Ababa located at Addis Ababa Observatory for the years between 1980 and 2009 were utilized to calculate monthly and annual average temperature. The computed average maximum and minimum temperature is presented in Table 3.2 below.

Table 3.2 Monthly and Annual Average Maximum and Minimum Temperatures of Addis Ababa

Station	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Ann total
Akaki Mission	26.5	27.6	28	27.8	28.2	25.9	24.2	24	25.15	26	25.8	25.9	17.06
AA observt	24.3	25	25.7	24.9	25.3	23.6	21.2	21	22	23.1	23.3	23.1	23.54
Bole station	23.85	24.7	26	25.1	25.1	23.2	21.1	21	21.7	23.15	23	23	23.24
Entoto	19.9	21.1	20.6	20	19.7	17.8	16.2	16.4	16.7	18.2	19.4	18.8	17.06
Mean Monthly	23.63	24.6	25	24.45	24.57	23.7	20.67	20.6	21.4	22.61	22.9	22.7	22.56

As can be observed in Table 3.2, the highest monthly average maximum temperature occurs in the months of March with 25^0C and the lowest is in the month of August with 20.6^0C. The monthly average record of temperature shown for each station indicates that the average annual temperature at Entoto, Addis Ababa Observatory, Addis Ababa Bole, and Akaki stations were 17^0C, 24^0C, 23^0C and 26^0C, respectively. Thus, the city characterized by an about of 23^0C mean annual temperature. Furthermore, Entoto station which is located at a higher elevation records smallest amount of average annual temperature. On the contrary, being at lower elevation, Akaki station records the highest amount of average annual temperature. This shows that, like the rainfall, there is also variation in the amount of temperature within Addis Ababa with differences in altitude.

3.6 Hydrogeology

Hydrogeological studies as part of engineering geological and geotechnical investigations are very crucial as sub-surface water is often a critical factor in various engineering works. (Kebede Tsehayu and Taddesse Hailemariam, 1990).

The ground water circulation and the down ward flow of pollutants through rocks and soils are depending on the hydrological characteristics of the material more specifically hydraulic properties such as porosity, permeability, transmissivity etc. the origin, flow and chemical constituent of groundwater are controlled by the lithology, distribution, thickness and structure of hydrogeological units through which it moves (UNESCO, 1972). Moreover, the stresses due to tectonism and weathering govern the hydro geochemical characteristics of earth materials (tamiru et al., 2005) therefore; to identify and select the best suitable site for the solid waste disposal it is necessary to describe the earth material occurring in the study area with a particular reference to their infiltration capacity. Volcanic rock mainly basalts, rhyolites, trachytes, scoria; trachy-basalts, welded and UN welded tuffs are the dominant rock outcrops in the area. Beside, unconsolidated materials of different origin also occurred in the study area. These rocks are the major groundwater supply for large parts of Addis Abeba (tamiru et al., 2005)

The study area is characterized by alternate eruption of basic and acidic lava flows from different centers.in between successive lava flows physical disintegration and chemical decomposition of rocks exposed at the surface; subsequent erosion and deposition; and tectonic activity taken place that has modified significantly the geomorphologic set up of the area. The main porosity groups identified are fracture porosity and interstitial porosity.

3.7 Soil

The soil development in the study area is mostly due to the physical disintegration and chemical decomposition of volcanic rocks. The weathering products are either remain in place and form residual soils or transported and deposited in the area of Addis Abeba. Meanwhile, the difference observed in the type and development of soils in the city is mostly depends on the topography, parent materials and the degree of weathering (Kebede, 1990).

Although there is significant difference in the degree of weathering on the slopes, mostly soils are highly eroded and result in thin soil cover.in the localities where the topography is plain to gentle (central and southern part) of the area is covered by thicken soil profile. The type of parent material and the length of time to which the parent material is subjected to weathering, control the variation in the thickness of the soil. Thus, old basic and acidic rocks that outcrop in the central, western and southwestern parts of Addis Abeba are weathered and form thick soil profile. In places where young basalt and welded tuffs occur, the thickness of the soil cover is reduced. The grain size distribution made by Kebede Teshayu (1990) showed that the residual soil in central parts, Gulele and Kolfe regions have 62% clay, 33% silt and 5% sand. In some localities reddish brown soil with a thickness of more than 10 meter is commonly seen.

Moreover, according to Lulseged Ayalew (1990) studies the residual has a thickness of about 2-6 meters and characterized by very high clay fraction with respect to silt and sand. The color varies from reddish brown to black depending on the type of parent materials.

The detrital materials that are derived from elevated area of Entoto, Wechecha, Furi and Yerer are transported and deposited in the piedmont and along the stream courses of Addis Abeba.it covers most part of Mekanisa, Ayere Tena, Kaliti, Akaki, Lideta, and Bole .the soil is black in color and the thickness varies from place to place primarily depending on the slop of the area. samples taken from Mekanisa are has 76% clay,22% silt,and 2% sand.it shows extremely high plasticity and very high degree of swelling (Kebede Tsehayu,1990).the work identified 46%silt,34%clay and20% sand in alluvial soil collected near Addis Abeba Bole Airport.

In areas where there is great contrast in the topography colluvial soils are found. These are loose and incoherent deposits, consisting of fine to coarse grain particles. Colluvial soils are mainly located at the foot slopes of northeastern part of Entoto silicics and other few places.

3.8 Expansive soil in the study area

The black and grey soils found in the eastern and southern part of Addis Ababa are highly expansive and there is no distinction between the heaving characteristics of the grey and black soils. It is well reported that expansive soils cover extensive areas in Addis Ababa (Sisay Alemayehu. (2004).The thickness of black cotton soil varies from place to place from 2m to 10m. The highest thickness is found in Bole area and in Beklo Bet area it is about 5m thick (Kebede, 1990; as cited in Dejenie Abere, 2017). (Figure 3.2).

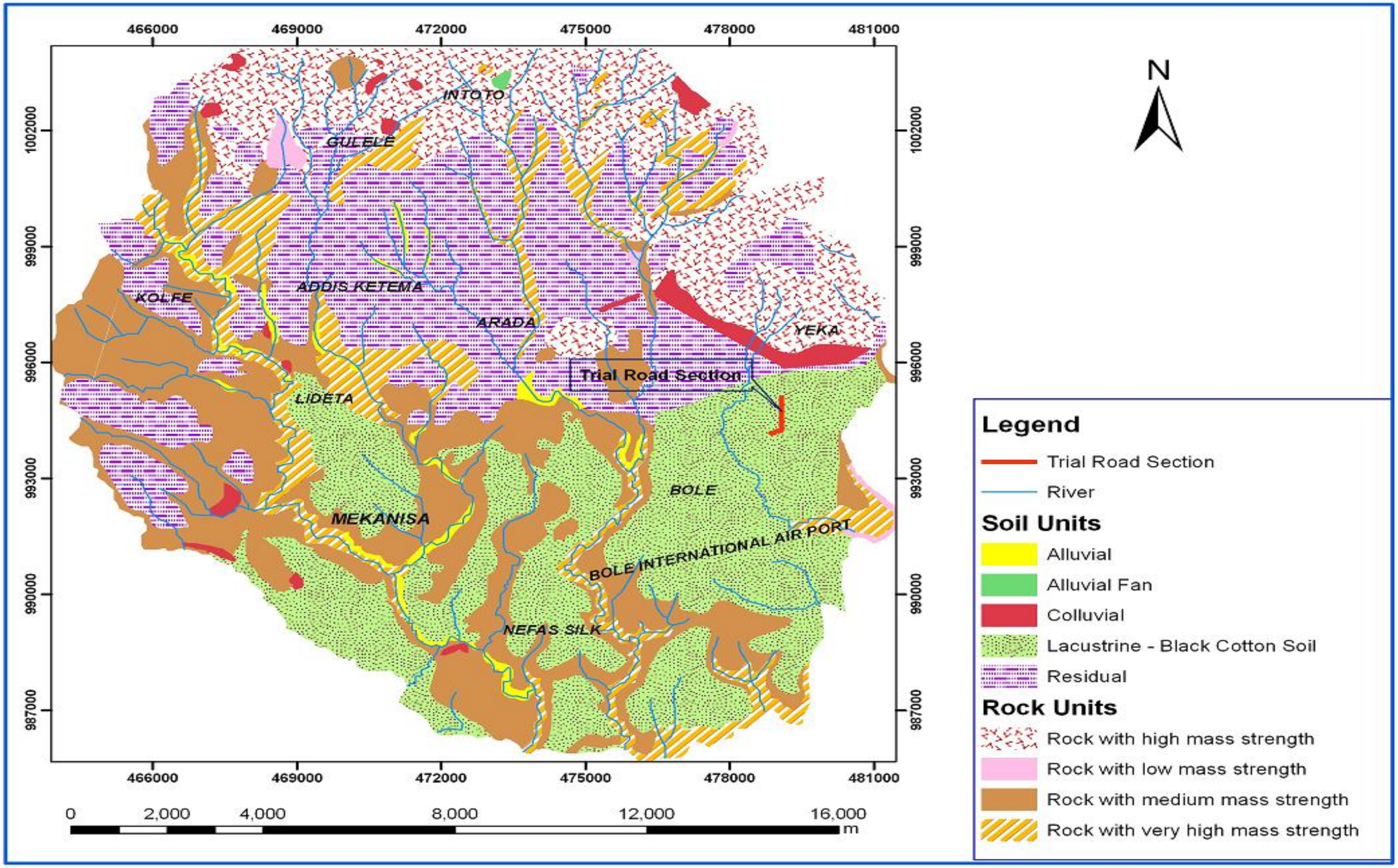

Figure 3.2. Engineering Geological Map of Addis Ababa (after Kebede Tsehayu et al., 1990)

CHAPTER FOUR

CHARACTERIZATION OF EXPANSIVE SOILS

4.1 Introduction

Expansive soils challenge to infrastructure development, particularly light loaded and shallow founded structures. Engineering geological characterization helps to identify the physical and engineering geological property of such soils, define their associated effects and assist the efforts in working with such soils. Thus, characterization of these soils before constructing the foundation is imperative for the safety of overlying building.

For the purpose of characterization, a large quantity of drilling and laboratory testing data, from ongoing projects, have been procured and analyzed. In order to perform a meaningful statistical analysis, borehole and laboratory data was collected from one company namely ARCON Design Building PLC. Geotechnical database was then developed from data obtained from this company.

Accordingly, index and engineering properties were determined from laboratory tests, and further analysis and interpretations have been made by integrating all laboratory tests with in- situ testing data.

This chapter mainly focuses on physical, index and engineering properties of the expansive soils of the study area, their distribution, genesis and basically to characterize the expansive soils.

4.2 Distribution and Genesis

4.2.1 Distribution of expansive soil

The geographical distribution of soils that are known to have expandable clay minerals which can cause damage to foundations and structures. It also include soils that have a clay mineral composition which can potentially cause damage.

4.2.2 Genesis of expansive soils

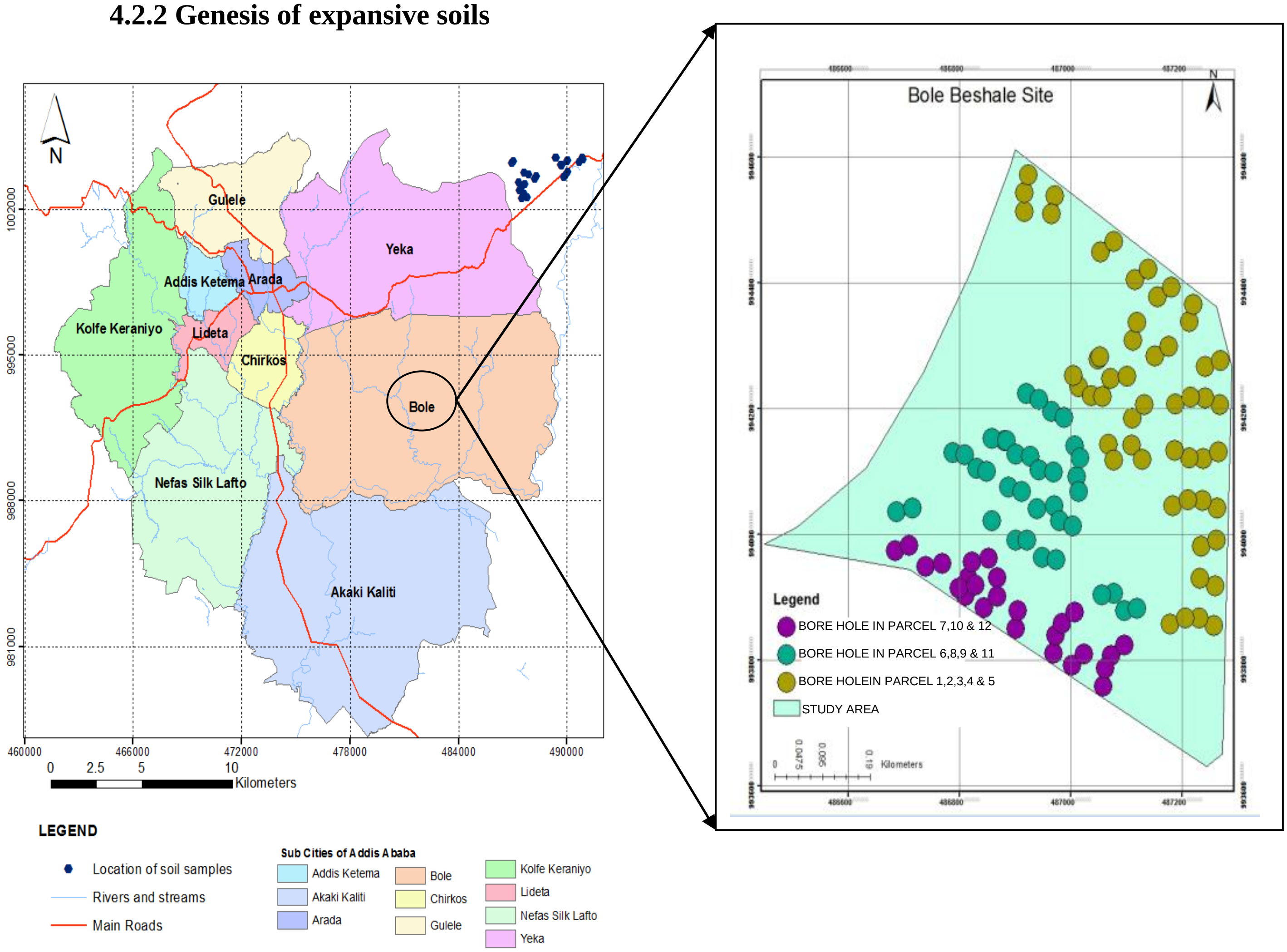

Figure 4.1 Distribution of borehole data within the study area

The origin and mineralogical composition of black and red clay soils of Ethiopia have been studied by (Morin and Parry, 1971). Black clay soils contain montmorillonite as the principal clay mineral with accessory kaolinite and halloysite whereas red clay soils contain kaolinite and halloysite as the principal clay with accessory montimorillonite. Such soils formed over Tertiary to Recent basaltic rocks which cover the central, western, north-western, south- western and some parts of southern and eastern Ethiopia. The associated volcanic rocks are the main source for the black clays formation. They often occur in flat and gently sloping landscapes such as on the highland plateau, low land flood plains and valley floors (Morin and Parry, 1971).

4.3 Soil distribution in the study area

To know the soil distribution of the study area, soil cross sections have been taken by connecting bore holes across the selected study area. Three geotechnical cross sections were taken from the three transect lines. Those sections were selected in a way that could help to see the variation of soil layers and thickness against topography.

The section shows that the upper most layer of the study area is dominantly black cotton soil with minimum depth 0.50m on bore hole B2-06-BH1 and maximum depth 3.50m on bore hole B11-53BH1. Silty clay/clayey silt layer is also available on minimum depth of 3.50m on bore hole B208-BH1 and maximum depth of 18.60m on bore hole B5-25BH2. And Silty sand/sandy silt layer is also available minimum depth of 3.05m on bore hole B8-37BH2 and maximum depth 19.50m on bore hole B2-06-BH1.

Section 1-1 through B8-36BH2, B8-37BH2, B11-53BH1 and B208-BH1

As shown in the figure 4.2 the first layer along this section is black cotton soil with a minimum 0.8meter depth and a maximum depth of 3.5meter thickness. Next to the first layer silty clay/clayey silt is highly dominant in this bore holes with a minimum 3.50meter depth and a maximum depth of 13.55meter thickness. And the other layer in this geotechnical cross section is silty sand/sandy silt with a minimum depth of 3.05meter and maximum depth of 13.40meter thickness.

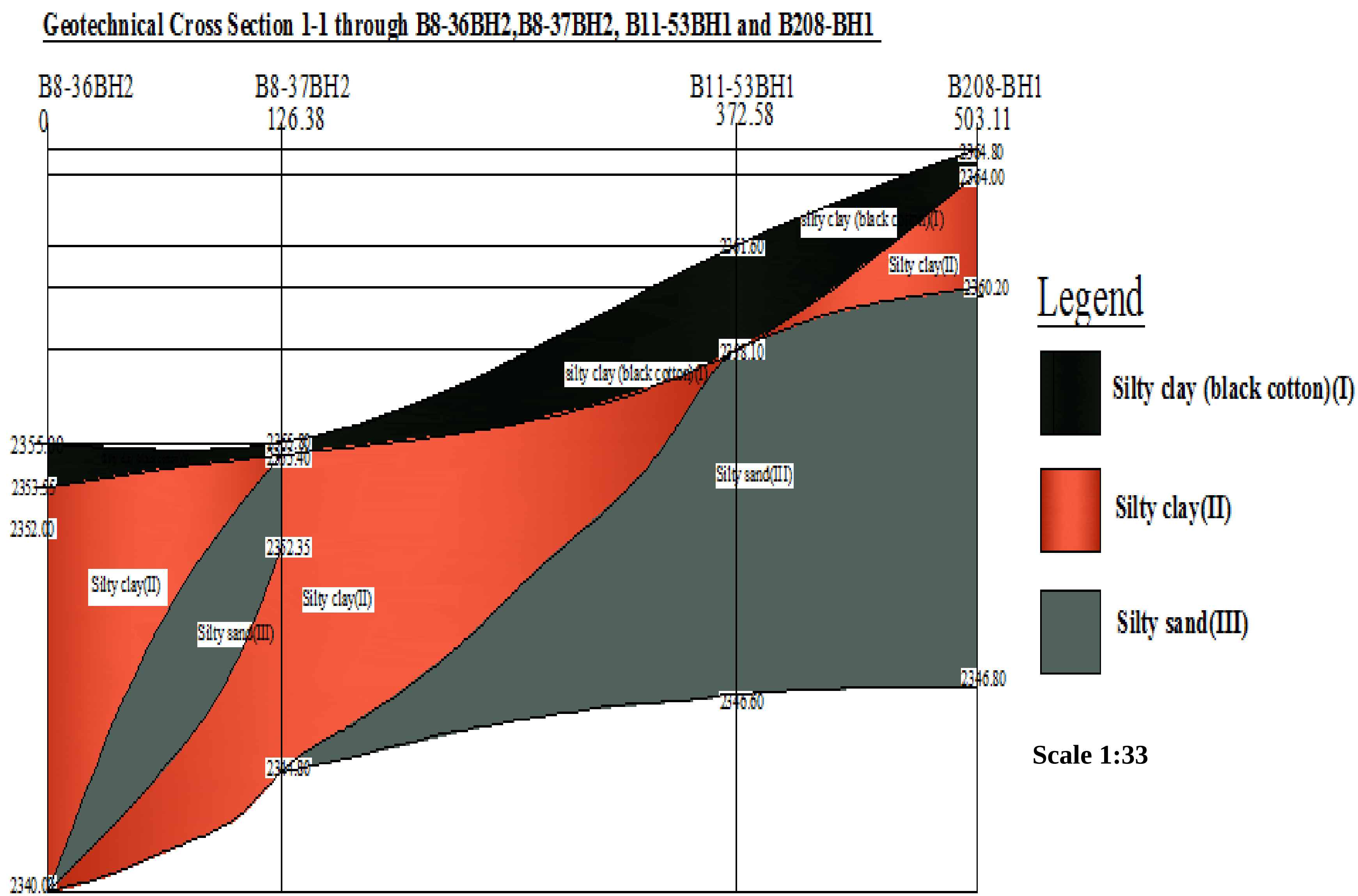

Figure 4.2. Geotechnical Cross- Section 1-1

Section 2-2 through B7-33BH2, B9-44BH1, B10-48BH2 and B213-BH1

As shown in the figure 4.3 the first layer along this section is black cotton soil with a minimum 1.30meter depth and a maximum depth of 1.88meter thickness. Next to the first layer silty clay/clayey silt is highly dominant in this bore holes with a minimum 8.20meter depth and a maximum depth of 16.00meter thickness. And the other layer in this geotechnical cross section is silty sand/sandy silt with a depth of 10.50meter thickness.

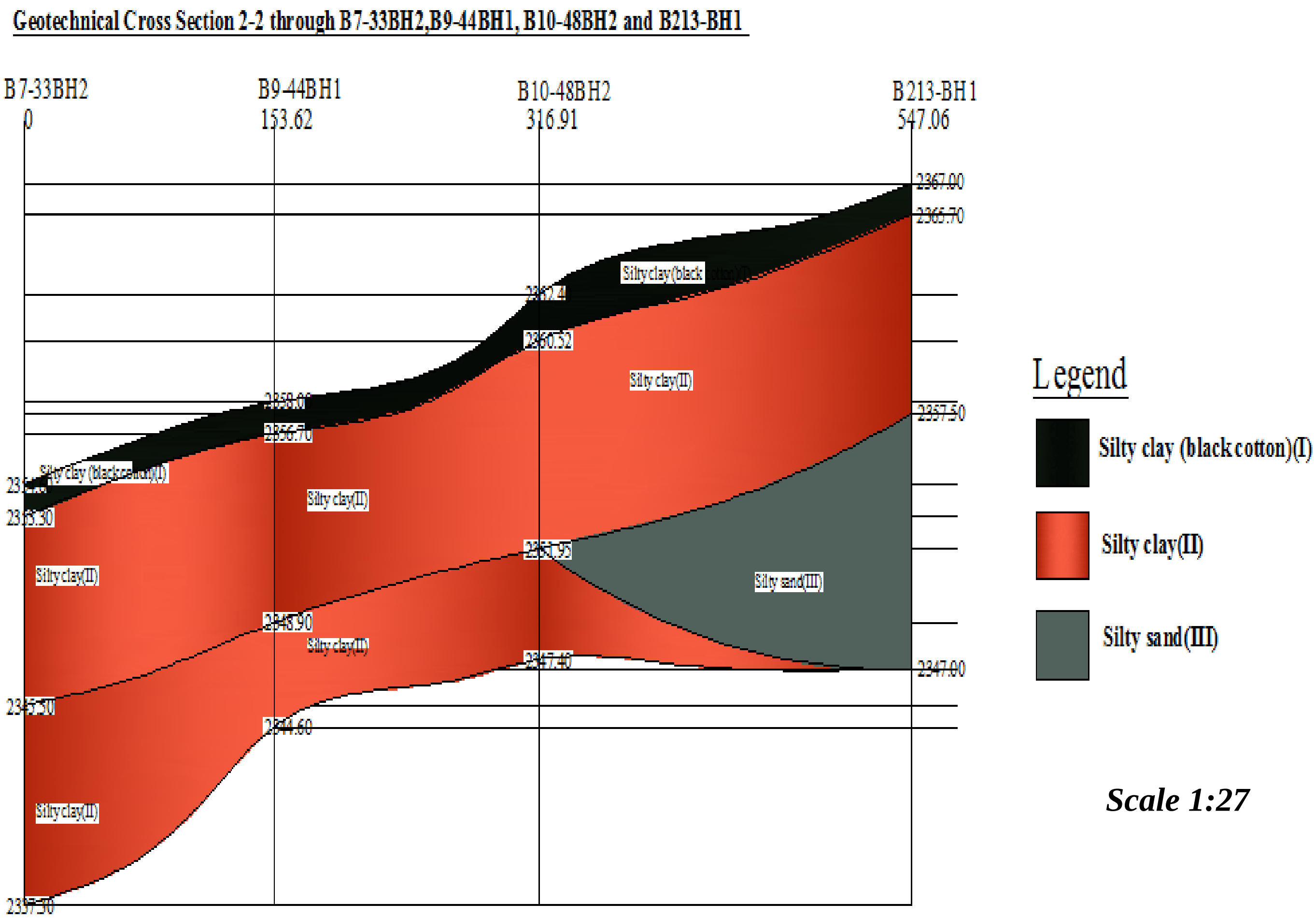

Figure 4.3. Geotechnical Cross section 2-2

Section 3-3 through B5-25BH2, BH10-45BH1, and B2-06-BH1

As shown in the figure 4.4 the first layer along this section is black cotton soil with a minimum 0.50meter depth and a maximum depth of 1.40meter thickness. Next to the first layer silty sand/sandy silt is highly dominant in this bore holes with a minimum 13.90meter depth and a maximum depth of 19.50meter thickness. And the other layer in this geotechnical cross section is silty clay/clayey silt with a depth of 18.60meter thickness.

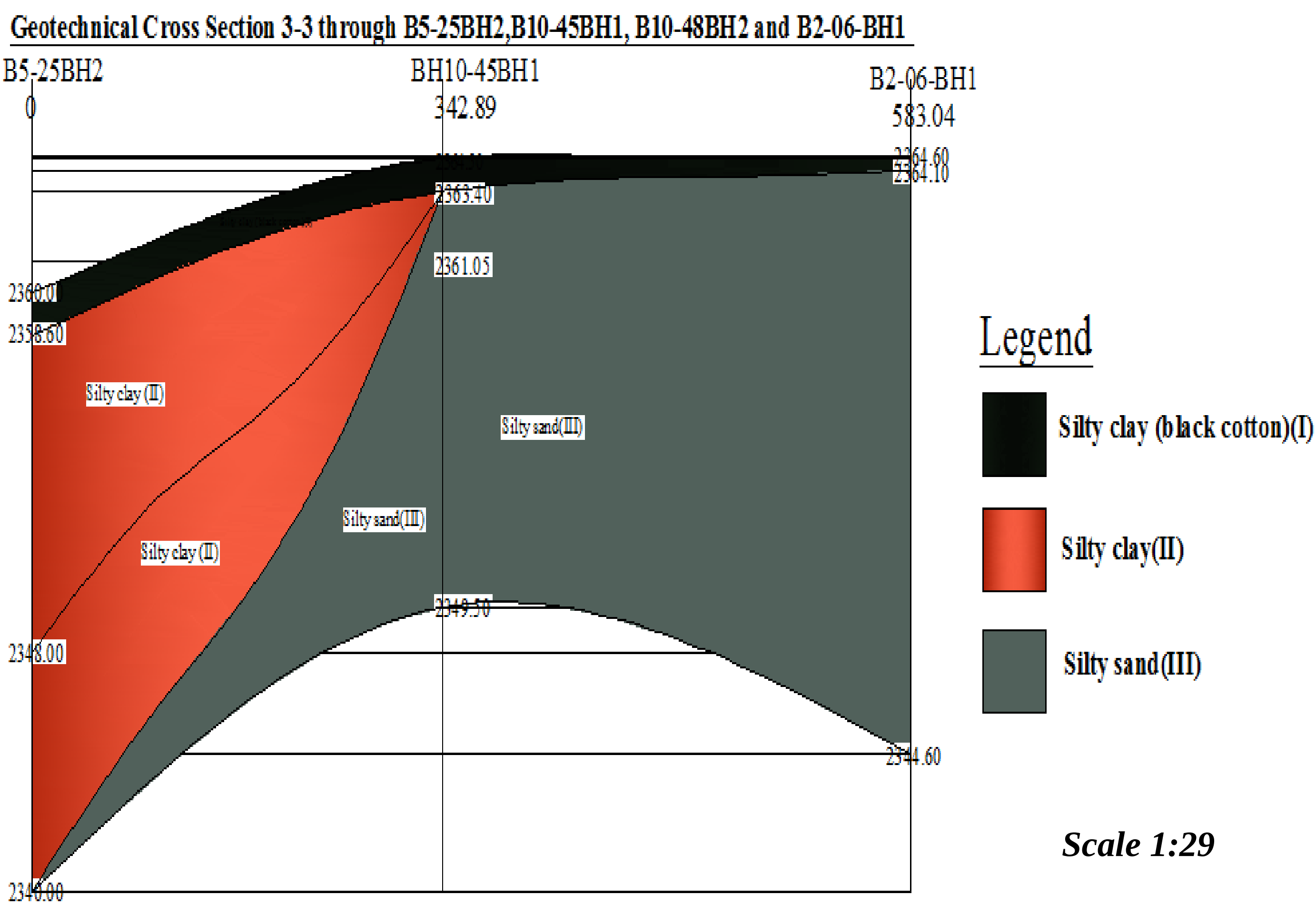

Figure 4.4 Geotechnical Cross Section 3-3

Generally in geotechnical cross sections it is observed that silty clay/clayey silt which is 55% exhibits highly dominant over sandy silt/silty sand which is 45% in the study area.

4.4 Index Property

Index property is a property that helps in distinguishing the characteristics of a soil. Soil grain property and soil aggregate property are two main categories under this term (Murthy, 2009). Soil grain property is based on the individual grains and depends on size, shape and mineralogical characteristics. Soil aggregate property, on the other hand is based on the property of the soil mass as a whole. Atterberg limit test, grain size analysis, specific gravity and free swell tests are among the tests which show the index property of a soil (Bowles, 1984).

Laboratory index test results namely particle size, grain size distribution, Atterberg limits, and free swell are presented in the following paragraphs.

Grain Size Distribution

Grain size distribution analysis, as one of the index property, divides soil into two distinctive groups, namely cohesion less and cohesive soil (Bell, 2007). Soil particles, which are coarser than 0.075 mm, are generally termed as cohesion less and the finer ones like silt and clay are considered fine grained. These can be done by sieve analysis and sedimentation analysis (Hydrometer analysis) (Budhu, 2011)

For the present study, sieve analysis, was conducted on two hundred eighty four soil samples to determine the combined percentage of silt and clay materials passing sieve No 200 (0.075mm) used in the classification of the soil type and the grain size distribution of this fine fraction is determined by means of hydrometer analysis method (Appendix 1).

The study area soils are ranging from 42 to 110% with an average value of 76% passing in # 200 sieves it is silty clay as shown in table 4-1.

Table-4.1 Physical and index properties of the study area soils

Physical and index property	Range	Mean
200 sieve pass (%)	42-110	76.00
Liquid limit (LL) (%)	32-99.70	65.85
Plasticity Index (PI) (%)	10.50-53.80	32.15
Free swell (%)	31.27-105.42	68.34

Atterberg Limits

When a clayey soil is mixed with an excessive amount of water, it may flow like a semiliquid. If the soil is gradually dried, it will behave like a plastic, semisolid or solid material, depending on its moisture content. The moisture content, in percent, at which the soil changes from a liquid to a plastic state is defined as the liquid limit (LL). Similarly, the moisture content, in percent, at which the soil changes from a plastic to semisolid state and from a semisolid to a solid state are defined as the plastic limit (PL) and shrinkage limit (SL), respectively. These limits are referred to as Atterberg limits (BRAJA DAS, 2001)

Atterberg limit corresponds to values of moisture content where the consistency of the soils change as it is progressively dried from slurry (liquid state). Plasticity is the response of a soil to changes in moisture content. When water is added to a soil, it changes its consistency from hard to soft; the soil is said to exhibit plasticity (Budhu, 2011). The Atterberg limits of measurement adopted in this study to describe the consistency of soils include the liquid limit (LL) and the plastic limit (PL). The liquid limit is the moisture content at which a soil passes from the liquid to the plastic state, while the plastic limit represents the moisture content at which a soil passes from the plastic to the solid state and becomes too dry to be in a plastic condition. The plasticity index (PI) is the moisture content which range between the plastic and liquid limits, and serves as a measure of the plasticity of the clay soil (Nelson and Miller, 1992).

For the present study, physical and index properties of the study area soils was conducted on two hundred eighty four soil samples (Appendix 1).

The plot of plastic limit against liquid limits can be used to detect the potential swell using Casagrande's plasticity chart (Fig 4.5). Soils that plot above the A-line are clays and those, which plot below it, are silts. The U-line indicates that the upper bound for natural soils. For the present study the collected soil samples fall above and below A- Line which is high plastic clay (CH) and high plastic silt (MH) respectively.

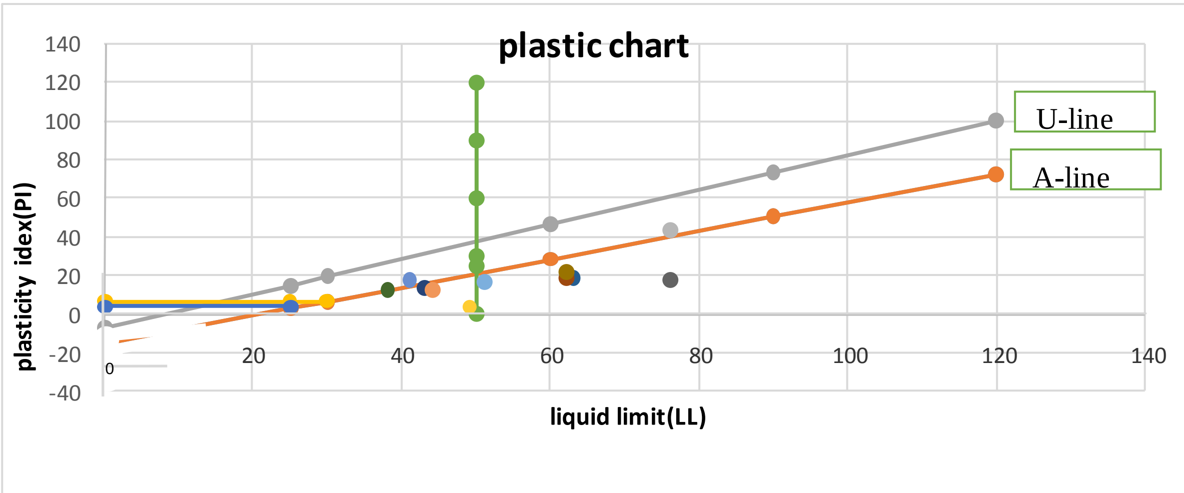

Figure 4.5 Plasticity chart for the soils of the study area according to USCS

In general, soils that exhibit plastic behavior over wide range of moisture content. Those with high liquid limits have greater potential for swelling and shrinking (Nelson, 2010).

The Atterberg limits had been correlated very well with the geotechnical characteristics of fine grained soils and are therefore very valuable in soil classification. Holtz and Gibbs (1956) demonstrated that the plasticity index and liquid limit are useful indices for determining the swelling characteristics of most of the clays. Soils with plasticity index that is above 35 and liquid limit greater than 70 have very high swell potential.

According to Seed et al. (1962) plasticity index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil. But, it should be noted here that high index property doesn't necessarily mean high swelling potential while the converse may be true.

From this study the liquid limit (LL) values are ranging from 32% to 99.7% with an average value of 65.85% and plasticity index (PI) values ranging from 10.50% to 53.80% with an average value of 32.15% (Table-4.1) this indicates that the soil exhibits high swelling potential (table 4.2)

Using Holtz and Gibbs (1956) the plasticity of the soil can be determined. Accordingly, the soils of the study area are classified as high to very high swelling potential. The values in Table 4.2 are proposed to relate swelling potential with index property.

Table 4.2 Classification of swelling potential based on plasticity (Holtz and Gibbs, 1956)

Classification of Swelling potential	Liquid Limit (LL) %	Plasticity Index (PI) %
Low	20-35	<18
Medium	35-50	15-28
High	50-70	25-41
Very High	>70	>35

The capacity and degree of clay soils to swell when moisture content of their environment increases is referred as a swelling potential (Nelson, 2010). The free swell test is one of the most commonly used simple test for estimating the soil swelling potential. According to Holtz & Gibbs (1956) the amount of free swell serves to be indicative of the probable swelling and/or expansive behavior of clay soils. It is considered as a measurement of volume change in clay upon saturation.

Chen (1988) presented a single index method for identifying expansive soils using only plasticity index (Table 4-3)

Table 4-3 Expansive soil classification based on plasticity index Chen (1988)

Swelling potential	Plasticity Index (%)
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

According to Chen (1988) as shown in table 4.4 below the whole soil sample test results showed that very high swelling potential with a single plasticity index. And in previous table 4.1 mentioned that the mean plasticity index of study area is 32.15% plasticity index; it indicated that they have high to very high swelling potential.

Table 4.4.Classification of Soil Sample Test Results Based On Plasticity Index Values

Location	LL (%)	PI (%)	Swelling potential
B211-BH2	74	38	Very high
B211-BH2	79	41	Very high
B9-39BH1	82	43	Very high
B9-39BH2	82	46	Very high
B9-42BH2	86	49	Very high
B9-42BH2	88	51	Very high
B9-42BH2	92	52	Very high
B9-42BH2	98	54	Very high
B9-42BH2	98.8	58	Very high
B10-48BH1	102	62	Very high
B10-48BH1	104	63	Very high
B10-48BH2	110	66	Very high

Classification of swelling potential based on free swell is summarized in Table (4.5). For the collected two hundred eighty four soil samples, the free swell values ranges from 31.27% to 105.42% with an average value of 68.34% and this shows that medium swelling capabilities, being characterized.

Table 4.5 Free swell classification of clay soils (Holtz and Gibbs, 1956).

Free swell value (%)	Free swell classification	Swelling Potential
<50	Low	Low
50-100	Medium	Medium
100-200	High	High
>200	Very high	Very high

4.5 Soil Classification

Soil classification system is a universal language which all the geotechnical engineers and engineering geologist understand, where soils of similar behavior are grouped together, and systematic and rational ways are proposed to classify and describe them. The use of such standard and precise term eliminates any ambiguity in communicating the soil characteristics (Robert, 2001).

The most widely used soil classification systems for engineering purposes are American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification System (USCS). The AASHTO system of soil classification comprises seven groups of inorganic soils from A-l to A-7 with 12 subgroups in all. The system is based on particle-size distribution, liquid limit and plasticity index. On the other hand the Unified Soil Classification System (USCS) is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil into three major divisions: coarse-grained soils, fine grained soils and highly organic soils. The AASHTO classification system is quite popular in road works (Lucian, 2006).

For the present study Unified Soil Classification system (USCS) was adopted; and both the light and dark grey soils of the study area soils fall above the A- line and below A-line which is under high plastic silt (MH) and high plastic clay (CH) soil type respectively (Figure.4.5).

4.6 Engineering Properties

The behavior of every foundation depends primarily on engineering characteristics of the underlying materials (Budhu, 2011). The most important engineering properties of soils for foundation Analysis are (cohesion and internal friction), and compressibility properties (such as the compressibility index).

Table 4.6: Engineering properties of the study area soils

Engineering property	Range	Mean
Unit weight (ɤ) (KN/m3)	17.8-23.6	20.7
Void ratio (e) (%)	0.906-0.966	0.936
Coefficient of Compression (Cc)	0.156-0.183	0.170

Compressibility

Among the compressibility properties, compression index is by far the most important engineering property to estimate settlement of foundations. It is defined as the slope of void ratio versus logarithm of the applied load curve in one dimensional consolidation test graph (Budhu, 2011).

For the collected 6 (six) boreholes soil samples, consolidation test results, Compression index (Cc) of light to dark grey soils ranges from 0.127 to 0.354 (Table-4.7). Table 4.8 gives an estimation of compressibility class based on compression index (Lambe and Whitman, 1979); and as it is shown table 4.8 the soils are ranging from low to medium compressibility.

Table 4.7: Boreholes soil samples consolidation test results with swelling potential

Location	LL (%)	PI (%)	Cc	Approximate Compressibility	Swelling potential
B211-BH2	74	38	0.127	Low	Very high
B9-39BH1	82	43	0.190	Low	Very high
B9-39BH2	82	46	0.191	Low	Very high
B9-42BH2	86	49	0.205	Medium-high	Very high
B10-48BH1	102	62	0.308	Medium-high	Very high
B10-48BH2	110	66	0.354	Medium-high	Very high

Table 4.8: Degree of compressibility (Lambe and Whitman, 1979)

Compression index Cc	Compressibility class
< 0.2	Low
0.2-0.8	Medium-High
0.8-2.6	High
> 2.6	Very high

Settlement

One of the most important properties of soils for estimating the settlement is the void ratio. The void ratio also indicates the softness or stiffness of a soil. Void ratios of expansive soils in the study area from samples are collected and the values ranges from 0.906 to 0.966 (Table-4.6).

According to Bowles (1984) void ratio less than or equal to 0.35 indicates the most densest state while void ratio greater than or equal to 1 implies that the soil is loose or soft. This implies that light to dark clays in the study area are in the range of soft to firm state. The higher the void ratio the higher the compressibility or settlement potential, a soft soil is one that can be compressible to a higher degree because of its high void ratio.

CHAPTER FIVE

CONSOLIDATION AND SETTLEMENT OF EXPANSIVE SOIL

5.1 Consolidation

The compressibility of soil is indicated by its change in volume per unit load increment. Any structure built on the ground causes increase of pressure on the underlying soil layers. The soil layers being confined by the surrounding soil strata adjust to the new pressure mainly through deformation. The vertical compression of the soil mass under increased pressure is thus made up of the components of a compression of solid matter, which under usual loadings accounted for very small compaction; a compression of the pore fluid, which maybe considerable where the pores contain air, but negligible when the pores are completely filled with water; reduction of the pore space by expulsion of pore fluid, which forms the major component of the compression. Any structure with high porosity, is more compressible than a dense structure. When the pressure on a soil is increased in all directions, the volume decreases. A study of compressibility of soil is necessary to be able to forecast the probable settlement of structure on different types of soils (soil mechanics Alemayehu tefera and Mesfin Leikun 1999).

When a soil mass is subjected to a compressive force, like all other materials, its volume decreases. The property of the soil due to which a decrease in volume occurs under a compressive forces is known as the compressibility of soil. The compression of soils can occur due to one or more of the following causes; compression of solid particles and water in the voids, compression and expulsion of air in the voids and expulsion of water in the voids. Compression of solid particles is negligibly small. Compression of water in the voids is also extremely small, as the water is almost incompressible in the range of the stresses involved in the soil engineering. Therefore, the compression due to the first cause is not much significant. The compression of a saturated soil under a steady static pressure is known as consolidation. It is entire due to expulsion of water from the voids. It is similar to the action of squeezing of water from a saturated sponge under pressure. The soil behave as a saturated sponge. As the consolidation of soils occurs the water escapes. The solid particles shift from one position to the other by rolling, sliding and thus attain a closer packing. Settlement of a structure is its vertical, down ward movement due to a volume decrease of the soil on which it is built. In other words, the settlement is the gradual sinking of a structure due to compression of the soil below.

A study of consolidation characteristics is extremely useful for forecasting the magnitude and time of the settlement of the structure. (Soil mechanics and foundation engineering Dr.K.R. Arora, 2004). In practice, the flow of water and displacements which take place during consolidation are nearly always three–dimensional. However, the analysis of three-dimensional effects is extremely complex and rarely practicable (Davis and Poulos, 1965).

In this study, Terzaghi´s one–dimensional analysis was applied in estimating the magnitude of settlements and the rate at which they develop. In applying Terzaghi´s analysis, it would be usually assumed that soils subjected to consolidation are vertical, laterally confined, fully saturated, homogeneous and uniformly thick materials; in which Darcy´s law for the flow of water is valid and the coefficient of permeability and other soil properties remain constant during any one increment of applied stress. The analysis also holds for soils in which drainage and compression is one-dimensional in a vertical direction, and soil particles and water are practically incompressible. In addition, initial excess pore pressure due to application of a load should be uniform throughout the clay layer while one or both of adjacent strata to the clay layer are to be perfectly free-draining in comparison to the clay. The effect of the weight of the clay layer under consolidation should be also negligible.

The compression of soils under a given load is usually divided into three phases (Head, 1988) and includes initial compression, primary consolidation and secondary compression (Figure.5.1).

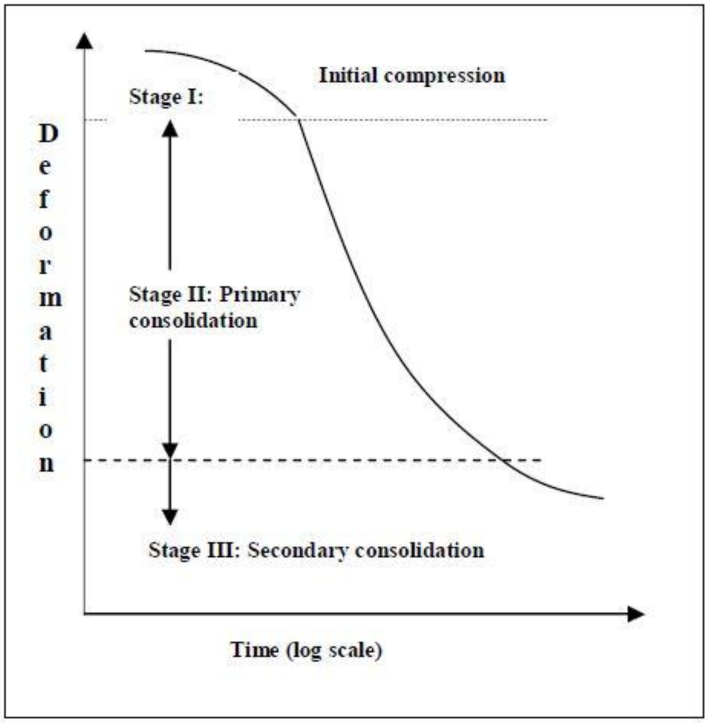

Figure 5.1 Log-time/Deformation curve, showing phases of consolidation (Head, 1988)

The initial compression takes place almost simultaneously with the application of a load increment in a laboratory test. It occurs due to expulsion of and compression of air in the voids. A small proportion may be due to elastic compression which is recoverable when the load is removed (Arora, 2004).

Primary consolidation is a time-dependent compression due to dissipation of excess pore pressure under loading. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as excess pore water, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the permeability of the soil (Jumikis, 1984). This phase is accounted for by the Terzaghi consolidation theory (Terzaghi, 1925) and relates closely to the theoretical curve for most clays. It is therefore used in many applications to make estimation of settlements. In fine grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability. As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles (Arora, 2004)

Secondary compression continues after the excess pore pressure of the primary consolidation phase has virtually dissipated. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the adsorbed water to the new stress system. In most inorganic soil, it is generally low (Arora, 2004).

5.2 Factors Affecting the Consolidation Characteristics of Clay Soils

The consolidation behavior of clay soil in its natural state is highly dependent on permeability and stress history (Jumikis, 1984).The expulsion of water from the voids of a saturated clay soil by an externally applied load in the consolidation process and the change in volume associated with such a process are essentially a hydraulic problem. Specifically, it is a problem of permeability of a soil to water. Therefore, the rate of consolidation depends on the permeability of the soil. The permeability of the soil by itself is a function of the soil type, size and shape of the soil particles (rounded, angular, or flaky), and thus, up on the size and geometry of voids (Arora, 2004).

The maximum stress to which the soil is subjected in the past influence the consolidation characteristics of the soil in its in-situ condition. The in-situ soil can be grouped in to two categories: Normally consolidated soils and Over-consolidated clay soil (Arora, 2004).

Normally consolidated soils: - A normally consolidated soil is one whose present effective overburden pressure on the in-situ prototype soil deposit is the maximum pressure to which the soil has ever been subjected at any time in the past history.

Over-consolidated clay soil: - Over consolidated clay is one which has been completely consolidated under a large overburden pressure in the past that is larger than the present overburden pressure.

5.3 Testing procedures

There are many oedometer tests that are used to measure consolidation properties. The most common type is the Incremental Loading (IL) test

5.3.1. Incremental Loading

The text Soil Mechanics in Engineering Practice describes a general procedure for the Incremental Loading test. A stiff confining ring with a sharp edge is used to cut a sample of soil directly from a larger block of soil. Excess soil is carefully carved away, leaving a sample with a diameter-to-height ratio of 3 or more. Porous stones are placed on the top and bottom of the sample to provide drainage. A rigid loading cap is then placed on top of the upper porous stone. This assembly is then placed into a loading frame.

Weights are placed on the frame, imposing a load on the soil. Compression of the sample is measured over time by a dial indicator. By observing the deflection value over time data, it can be determined when the sample has reached the end of primary consolidation. Another load is then immediately placed on the soil and this process is repeated. After a significant total load has been applied, the load on the sample is decreased incrementally. Using a load increment ratio of 1/2 provides a sufficient number of data points to describe the relationship between void ratio and effective stress for a soil.

ASTM International has a standard testing procedure for incremental loading: ASTM D2435-04. Other testing standards such as BS 1377:5, ASTM D3877, ASTM D4546 and AASHTO T216 provide procedures for conducting tests for determination of the consolidation characteristics of soils. Oedometer test set is required to perform the test. It is used to determine the consolidation characteristics of soils of low permeability. Tests are carried out on specimens prepared from undisturbed samples. Ideally, following would be needed to perform the Oedometer test.

- ✓ 1 Bench
- ✓ 3 Oedometers
- ✓ 3 Cells, either 50mm or 63.5mm, or 75mm
- ✓ 3 Dial gauges, either analogue, or digital
- ✓ 1 Weight set

5.4 Compressibility Characteristics

Compressibility Characteristics of soils forms one of the important soil parameters required in design considerations. Coefficient of compressibility (Cv), Coefficient of volume compressibility (mv) and Compression index (Cc). These coefficients are derived from consolidation tests and are indicative of the compressibility of soils. They serve to give estimates of the amount of settlement due to primary consolidation (Bowels, 1984).

The coefficient of compressibility (Cv) and the coefficient of volume compressibility (mv) are usually calculated for each load increment while the compression index(Cc) is derived from an e/ log p curve obtained by plotting voids ratio, e, against applied pressure, p, (log scale). In practice, (mv) is applied to over-consolidated clays and (Cc) to normally consolidated ones (Arora, 2004).

5.5 Results of Consolidation Tests

Void ratio

Results of voids ratio change of soils during loading and unloading stages are summarized in the form of voids ratio/ log pressure (e/log p) curves as shown in Figure (5.4). The curves occupy higher positions on the diagram due to their relatively higher void ratios. As a result, the soils would be expected to be compressible. In addition, unloading portions of the curves are steeper. This serves to show once again the higher potential swelling capabilities of the soil.

Results of oedometer consolidation tests in terms of voids ratio, compression coefficients and/ or indices over the loading range of 50-1600 KPa are summarized in Table (5.1).

Table 5.1 Derived parameters from results of oedometer consolidation tests for the loading range of 50-1600kPa.

Sample	Depth (m)	e_0	a_v (m^2/KN)	m_v (m^2/MN)	C_v (m^2/year)	Cc
B10-48BH1	5.0-5.5	0.9660	0.0001272-0.0001910	0.0764-0.3045	1.337-6.460	0.169
B211- BH2	7.0-7.5	0.9180	0.0001208-0.0001816	0.0726-0.2894	1.271-6.139	0.156
B9-42BH2	5.5-6.0	0.9132	0.0001255-0.0002752	0.1263-0.4250	2.840-11.922	0.156
B9-39BH1	5.0-5.5	0.9061	0.0000833-0.0001304	0.0837-0.2200	4.228-12.740	0.183

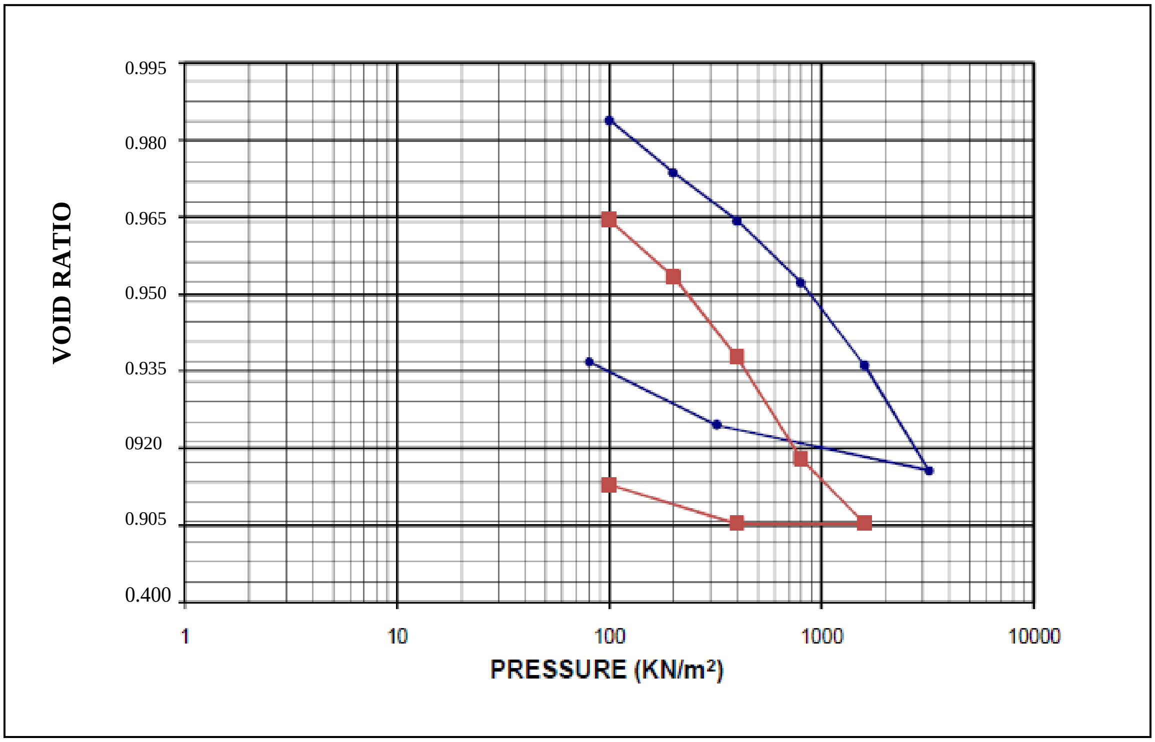

Figure 5.2. Voids ratio / log Pressure (e/ log p) curves of the selected soil samples(B211-BH2 & B10-48BH1)

Accordingly, tests on remolded samples of different dry density and the same moisture content have also indicated that the consolidation behavior of expansive soil is different from that of non-expansive clay soils (Figure 5.2). In non-expansive clay soils, a soil with larger dry density will have compression index smaller than the same soil of lesser dry density, whereas, in expansive soils, a different consolidation characteristics is observed. As can be seen from Table 5.1 test results of sample B211BH2 and sample B10-48BH1 gave compression index values of 0.169 and 0.156, respectively. A higher value of compression index is observed in the denser soil sample B211BH2. The observations made from the laboratory tests leads one to identify an additional factor, which governs the consolidation behavior of expansive soils. As can be seen from the test results, those factors that affect the swelling characteristics of expansive soils have also affected the consolidation characteristics of the expansive soil. `

Compressibility

Ranges of values of coefficient of volume compressibility, mv, obtained for the selected soils and are given in Table (5.2), where they have been used to classify the soils based on compressibility. Ranges of values of mv for other types of soil are also included for comparison purposes. Results in Table (5.2) show that the study area soils would generally exhibit medium to high compressibility over the loading range of 50-1600KPa.

Table 5.2. Guideline of compressibility classification of soils adopted to this study (after Lambe and Whiteman, 1979).

Category	Clay type/ Sample No	Coefficient Of volume compressibility (m_v) (m^2/MN)	Compressibility classification
Typical examples (after Lambe and Whitman, 1979)	Very organic alluvial clays and peats	>1.5	Very high
	Normally consolidated alluvial clays	0.3-1.5	High
	Fluvio-glacial clays and lake clays	0.1-0.3	Medium
	Boulder clays	0.05-0.1	Low
	Heavily over consolidated boulder clays	<0.05	Very low
	B10-48BH1	0.0764-0.3045	medium to high
	B211 BH2	0.0726-0.2894	Low to Medium
	B9-42BH2	0.1263-0.425	Medium to high
	B9-39BH1	0.0837-0.220	Low to medium

Ranges of values of coefficient of consolidation, Cv, obtained for the selected soil samples are given in Table (5.3). Typical ranges for other inorganic soils are included for comparison purposes. From the results, the rates of consolidation-settlement for the clay soil would be generally medium to high when externally loaded in the range of 50-1600KPa.

Table 5.3. Guideline of rates of consolidation-settlement of soils adapted to this study (after Lambe and Whiteman, 1979).

Category	Clay type/Sample No	Coefficient of consolidation (c_v) (m^2/year)	Rate of consolidation settlement
Typical examples for inorganic clays(after Lambe & Whitman, 1979)	High plasticity	0.1-1.0	Low
	Medium plasticity	1.0-10	Medium
	Low plasticity	10-100	High
	Silts	>100	Very high
	B10-48BH1	1.337-6.460	Medium
	B211 BH2	1.271-6.139	Medium
	B9-42BH2	2.840-11.922	Medium to high
	B9-39BH1	4.228-12.740	Medium to high

Results of compression index, Cc, obtained for the soils in this study are presented in Table (5.4) together with corresponding values of liquid limit and plasticity index and the result shows that the study area soils would generally exhibit low to medium compressibility. Based on compression index (Lambe and Whitman, 1979) clay soils could be approximately grouped into those of low compressibility, medium to high compressibility, high compressibility and very high compressibility; as shown in Table (5.3).

The results of correlation show that LL and PI, would be a better estimator of the compression index and compressibility of soils (Table 5.4).The strength of correlation between Cc and LL is very high (R^2 = 0.9493) and between Cc and PI is (R^2=0.9166)(Figure. 5.3 and 5.4). The relationships between compression index and liquid limit take the form of

$C_c = 0.0066LL - 0.3692$ --5.1

And the relationship between compression index and Plasticity index take the form of

$C_c = 0.0079PI - 0.1752$ ---5. 2

Table5.4 Results of compression indices correlated with Atterberg limits.

Location	LL (%)	PI (%)	Cc	Approximate Compressibility
B211-BH2	74	38	0.127	Low
B211-BH2	79	41	0.131	Low
B9-39BH1	82	43	0.190	Low
B9-39BH2	82	46	0.191	Low
B9-42BH2	86	49	0.205	Medium-high
B9-42BH2	88	51	0.206	Medium-high
B9-42BH2	92	52	0.209	Medium-high
B9-42BH2	98	54	0.301	Medium-high
B9-42BH2	98.8	58	0.302	Medium-high
B10-48BH1	102	62	0.308	Medium-high
B10-48BH1	104	63	0.312	Medium-high
B10-48BH2	110	66	0.354	Medium-high

Results of correlations and analysis show that Cc values generally increases with increasing liquid limit and plasticity index of soils (Table 5.4 Figure.5.3 & 5.4). This serves to suggest that compressibility of soils generally increases with increased plasticity, and vice versa

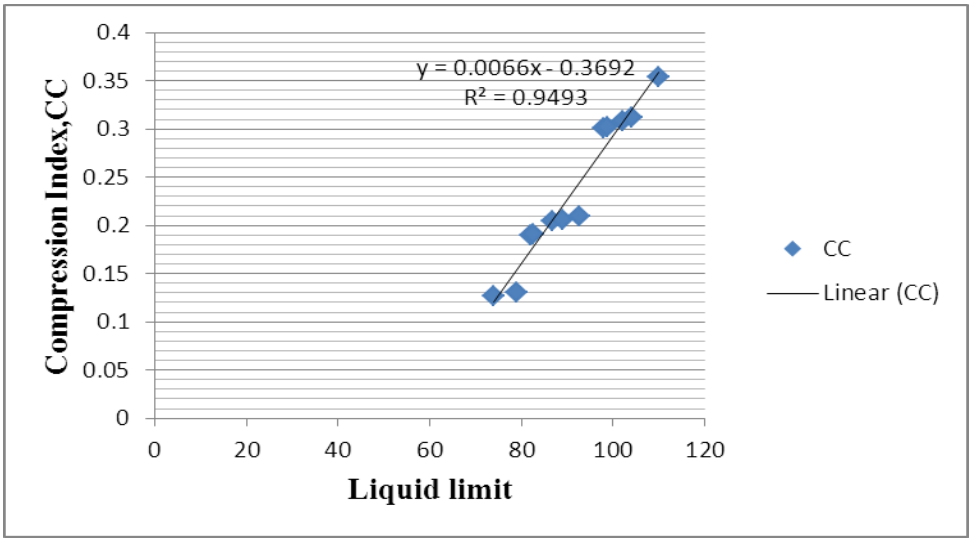

Figure 5.3: Correlation between liquid limit and compression indices

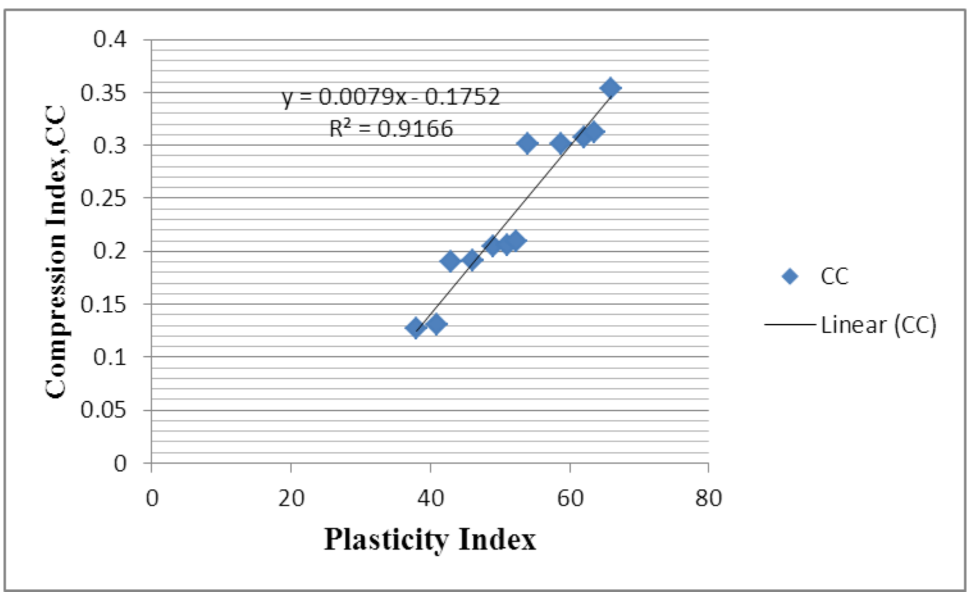

Figure 5.4: Correlation between plasticity index and compression indice

5.6 Consolidation Characteristics

Results of laboratory consolidation tests (Tables 5.2 and 5.3) show that expansive clay soils have been found to be characterized by generally medium to high compressible and, therefore, high consolidation settlements on external loading. The high rates of consolidation- settlement exhibited by these clays would also mean structural settlements occurring beyond the construction stage. Long-term instability problems would therefore be expected for structures constructed on these clays.

Results of oedometer consolidation tests obtained in this study could be used to solve problems related to foundations for light structures in the study area. The results would specifically assist in calculating the amount of settlement which would ultimately take place for the structure as a whole, while variations in long-term settlements and/ or differential settlements between isolated footings could also be estimated; by applying methods described by Terzaghi (1939).

According to Terzaghi (1939) differential settlements are usually more critical than overall settlement, and can cause tilting of a structure as a whole and/or distortions within the structure; and should therefore be kept within acceptable limits to avoid possible structural deterioration and damage we must use mat foundation to avoid differential settlement.

On the other hand, estimates of the rate of consolidation could be further used to give the duration of time within which structural settlements would be completed, either during or after construction. In situations of long-term settlements as may occur in expansive clays, settlement/time graphs could serve to show the duration of the most significant part of settlements, and this could be compared with the economic life of the structure. The settlement/time relationship would also assist in ascertaining possible development of unacceptable differential settlements in the long-term, after construction.

The use of results of oedometer consolidation tests is, however, limited when it comes to the estimation of rates of settlement of foundation soils. According to Rowe (1972) the possibility of under estimation is attributed to the small size of laboratory test specimens used which make it unrealistic to represent many of the natural features of soil fabric (fissures, pores, laminations, other discontinuities), and which have a profound effect and control on drainage and therefore the rate of consolidation-settlement. As a result, a given proportion of the ultimate consolidation-settlement is reached in a shorter time in real field situations, than that predicted from laboratory test data.

CHAPTER SIX

CONCLUSION AND RECOMMENDATION

6.1 Conclusion

This research is conducted in Addis Ababa taking a case study in Bole Beshale area to assess Consolidation Behavior of Expansive Soil.

In order to achieve the above mentioned objective, systematic literature review was undertaken that includes previous studies, various soil mechanics and foundation engineering books, manuals and research articles and online internet browsing. Thus, with this literature review a conceptual framework was developed and a feasible methodology was incorporated.

Identify and evaluate the sub surface soil profile by detail site investigation and determining the consolidation behavior of expansive soil on foundation play a key role for safe and economical design of engineering structure.

A geotechnical data base, a large quantity of drilling and laboratory testing data, from ongoing projects were taken and filtered out and utilized. The extracted laboratory and field investigation data were systematically organized and a geotechnical data base were created. The generated geotechnical data were used to derive new relevant correlations and mathematical model for the selected soil type. From this study the following findings are deduced;

Both the light and dark grey soils of the study area soils fall under the MH and CH soil class based on USCS respectively.

The soils show high contents of fine fraction, 42 to 110% with an average value of 76%, indicating that their physical and engineering behaviors are almost directly related to the type of their clay minerals.

Based on the available drilling geotechnical data, the thickness of expansive soil ranges from 1m to 4m, the maximum thickness was found in the study area. The clay exhibited slightly low to medium compressibility (m_v = 0.0726-0.425 m^2/MN) when externally loaded over a 50 –1600kPa range, as a result of its stiff and cohesive nature. In addition, unloading curves are more inclined indicating that the clays having more swelling capabilities and/or expansion tendencies. The clays would also exhibit high rates of consolidation-settlement (C_v= 1.271-12.74m^2/year) due to their relatively low permeability that tend to limit drainage and dissipation of pore-water pressures. Settlement and instability of structures located on these clays are then expected to persist beyond the construction stage.

The following relationships have been derived for the possible estimation of Compressibility Index, Cc, of the soils using laboratory determined LL and PI. $C_c = 0.0066LL - 0.3692$ between the calculated compression indices and laboratory measured compression indices were found to be strong. Result of correlations and analysis show that Cc values generally increases with increasing liquid limit and plasticity index of soils. This serves to suggest that compressibility of soils generally increases with increased plasticity, and vice versa.

Laboratory tests conducted on expansive soils in this research work have shown that the value of the consolidation parameters of the soil are smaller than the values for clay soil in other parts of the world. This means that the soil has relatively lesser compressibility and the rate of consolidation is also smaller as well.

6.2 Recommendation

Based on the cumulative outputs of this research the following recommendations are forwarded:

- ✓ The developed relations of compressibility indices using index tests needs verification with a wider range data base of standardized test results.
- ✓ To be recommended to initiate and establish a field monitoring program for the soils of the study area, aimed at monitoring of expansive soil movements and moisture changes. The scheme may take the form of monitoring and recording profiles of ground movements and moisture changes as well as providing general data on soils with time to design foundations on expansive soils and compare with models which may be developed in the laboratory to simulate and characterize possible field behavior of the soils.

References

Alemayehu Tefera and Solomon Yohannes (1986). Investigations on the expansive soils of Addis Ababa, *Civil Engineering Department.* **7**:1-9.

Alemayehu Tefera. (1999).Notes on Foundation of light-weight structures.Building Centre, Addis Ababa, September, 1965.

Altmeyer WT, (1956) Discussion of engineering properties of expansive clays. In: Proceedings, American Society of Civil Engineers, vol. 81, Separate No. 658; March 1955. p 218–34.

Arora, K.R. (2004). *Soil Mechanics and Foundation Engineering*. Standard Publishers, New Delhi, India, 886 pp.

Bell, F. (1999). Geological hazards: their assessment, avoidance and mitigation. **In:** *E & FN SPON*, Standard Publishers, London, Pp 648.

Blight, G. E. (1997). *Mechanics of Residual Soils*. A.A Balkema, Rotterdam, Netherlands, 589pp.

Bowles, J.E. (1984). *Physical and Geotechnical properties of soils*. McGraw Hill, New York, 1089 pp.

BS 1377:1975. Laboratory determination methods of engineering soil properties. British Standards Institution, London.

Budhu, M. (2011). *Soil Mechanics and Foundations*. John Wiley & Sons, Inc. New York,761 pp.

Chen, F.H. (1988). *Foundations on expansive soils*. Elsevier Science Publishers, Amsterdam, 562pp

Chen,F.H. (1998),Foundations on expansive soils, Elsevier Science Pulishers B.V

Craig, R.F (2004). *Craig's Soil Mechanics*, 7thed. Spon Press, London and New York, 760PP.

Das, B.M. (2011). *Principles of Foundation Engineering*, 7th ed., CT: Global Engineering, Stamford, 794 pp

Day, R.W. (2006). *Foundation Engineering Handbook.* McGraw hill, New York, 822 pp. Davis, E and Poulos, H. (1965). The analysis of settlement under three-dimensional conditions. **In:** *Symp. on Soft Groud Eng.,eds:* Inst. Eng. Australia, Brisbane 168pp.

Dejenie Abere (2017), Impact of Expandable Soil on Foundation of Building Structures: The Case of Condominium Houses in Addis Ababa, unpublished M.Sc. Thesis, Addis Ababa Science and Technology University, June 2017, pp.25

EMA, 1981, National Atlas of Ethiopia, Ethiopian Mapping Authority, Addis Ababa, Ethiopia.

(ERA) (Ethiopian Road Authority) (2002). Site Investigation Manual. Unpublished technical report, Addis Ababa, Ethiopia.

Geological Society (1997). Tropical residual soils, the Geological society, Engineering group working party revised report, London.

(GSE) (Geological Survey of Ethiopia), (1996). *Explanation to the Geological Map of Ethiopia*, Institute of Geological Surveys, Addis Ababa, Ethiopia, 69 pp.

Gilloth, J. E., (1962). *Clay in Engineering Geology*, Elsevier Publishing Company, USA, 567 pp.

Griffiths, 1972 as cited in Habtamu Solomon, 2011

Grim, R.E.(1953) *Clay Mineralogy,* McGraw-Hill Book Company, New York,740 pp

Habtamu Solomon (2011). Chemical Stabilization of Expansive subgrade soil performance evaluation on selected road section in northern Addis Ababa. Unpublished MSc Thesis, Addis Ababa University, Addis Ababa, Ethiopia, 97 pp.

Hana Tibebu. (2008). Study of Index Properties and Shear strength parameters of Lateritic soils in Welayita Sodo. Unpublished MSc Thesis, Addis Ababa University, Addis Ababa, Ethiopia, 119 pp.

Head, K. H. (1988). Manual of soil laboratory testing. Pentech Press Ltd., Plymouth, Devon 589pp.

Holland, J.E. and Richards, J. (1982). Road Pavements on Expansive Clays. *Australian Road Research*. **12** (1) : 295pp

Holtz, W. and Gibbs, H. (1956). Engineering Properties of Expansive Clays, *American Society of Civil Engineers*.**121**: 641-677.

Holtz, R. and Kovacs, W (1981). *An Introduction to Geotechnical Engineering*. Englewood Cliffs, NJ, 733pp.

Hussein A.Elarabi, 2014, Factors Influncing swelling behavior of expansive Soils, BRRJ vol.6 December 2009

Jones, J. and Holtz, W.G. (1973). Expansive Soils- The hidden disaster Civil Engineering. American Society of Civil Engineers (*ASCE).* **43**: 49-51.

Jumikis, A.R. (1984). *Soil Mechanics*. Robert E. Krieger Publishing Company, Florida, 791pp.

Kebede Tsehayu & Tadesse Haile Mariam, (1990*). Engineering Geological Mapping of Addis Ababa*, Ethiopian Geological Survey, Addis Ababa, Ethiopia, 84 pp. Lambe, W. and Whitman, V. (1979). *Soil Mechanics*, 2nd ed., Wiley, New York, 553pp.

Lucian, C. (2006). Geotechnical Aspects of Buildings on Expansive Soils in Kibaha, Tanzania: Unpubished preliminary Study.

Mitchell, J.K. (2005). *Fundamentals of soil behaviour*. 2nd ed., Wiley, New York, 437 pp. Mohr, P.A. (1962). *The Geology of Ethiopia*. University College of Addis Ababa Press, Asmara, 268 pp.

Morin, W. (1971). Properties of African Tropical Black Soils. **In:** *Fifth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, eds:*- Luanda, Angola, pp. 51-65.

Morin, W. and Parry, W. (1971). Geotechnical Properties of Ethiopian Volcanic Soils, *Geotechnique*. **21**: 223-232.

Murthy, V. (2009). *Principles and practices of soil mechanics and foundation engineering*. Marcel Dekker Inc., New York, 1029 pp.

Murray, H.H. (2007). *Applied Clay Mineralogy*. Elsevier scientific publishing company, USA, 780 PP.

Nelson, S. (2010). Weathering and Clay Minerals, Lecture notes, Tulane University, USA.

Nelson, D. and Miller, J. (1992). *Expansive Soils: Problems and Practice in Foundation and Pavement Engineering*. John Wiley & Sons, Inc, New York, 573pp.

Netterberg, F. (2001). Addis Ababa-Jimma road project preliminary report on cracking problem: Unpublished Report Ethiopian Roads Authority, Addis Ababa.

Robert, W.B. (2001). *Practical Foundation Engineering Handbook*. The McGraw Hill Companies Inc, New York, 1152 pp.

Ross, G. (1978). Relationships of Specific Surface area and clay content to shrink-swell potential of soils having different clay mineralogical compositions. *Canadian journal of soil science*. **58**:159-166.

Rowe, P. W. (1972). The relevance of soil fabric to site investigation practice. *Geotechnique*, **22**, No. 2

Seed, H.B., Woodward, R.G. and Lundgren, R. (1962). Prediction of Swelling Potential for compacted clays, *American Society of Civil Engineers Journal of Soil Mechanics and Foundation Engineering*. **88:**53-87.

Sisay Alemayehu. (2004). Assessment of Damages of Buildings Constructed in Expansive Soil Area of Addis Ababa. Unpublished MSc Thesis, Addis Ababa University, Addis Ababa, Ethiopia, 78pp.

Tamiru Alemayehu, Tenalem Ayenew, Dagnachew Legese, Yirga Tadesse, Solomon

Terzaghi, K. (1925). *Erdbaumechanik auf bodenphysikalischer Grundlage*. Deuticke, Wien, 580pp.

Terzaghi, K. (1939). Soil Mechanics- a new chapter in engineering science. *Inst. Civ. Eng*.**12**, No. 7.

Terzaghi, K. (2005). *Theoretical Soil Mechanics in Engineering Practice.* John Wiley and Sons, New York, 510 pp.

Appendices

Appendix 1: Results of index test on disturbed soil

on parcel 1,2,3,4 & 5										
BH ID	Depth, m	Lab. Description	Atterberg limit		% Gravel	% Sand	% Silt	FS %	GS	Group symbol
			LL	PI						
B1-01 BH1	5.00 - 5.50	High plastic clayey SILT	63	19	NIL	2	98	20	-	MH
B1-01 BH1	9.00 - 9.50	High plastic sandy SILT	62	19	1	13	86	30	-	MH
B1-01 BH1	16.00 - 16.50	High plastic silty GRAVEL with sand	76	18	37	20	43	30	-	GM
B202 BH1	7.00 - 7.50	Low plastic sandy SILT	46	13	NIL	33	67	NIL	2.47	MH
B202 BH1	10.50 - 11.00	High plastic clayey SILT	90	28	NIL	4	96	30	2.41	MH
B202 BH1	13.70 - 14.20	High plastic clayey SILT	92	41	NIL	2	98	80	-	MH
B202 BH2	4.15 - 4.65	Non plastic silty SAND	NP	NP	NIL	59	41	NIL	-	-
B202 BH2	8.50 - 9.00	Low plastic sandy SILT	42	14	NIL	21	79	NIL	-	ML
B202 BH2	14.00 - 14.50	Low plastic clayey SILT with few sand	50	15	8	12	80	10	2.45	ML
B303 BH1	6.50 - 7.00	Non plastic clayey SILT	NP	NP	NIL	8	92	10	2.34	-
B303 BH1	10.00 - 10.50	High plastic clayey SILT	62	22	NIL	2	98	NIL	2.53	MH
B303 BH1	14.50 - 15.00	Low plastic sandy SILT	43	14	1	47	52	40	2.39	ML

B303 BH2	12.00 - 12.50	Low plastic silty SAND	NP	NP	3	62	35	-	2.49	-
B303 BH2	16.50 - 17.00	High plastic clayey SILT	67	17	NIL	2	98	20	-	MH
B404 BH1	5.50 - 600	Non plastic silty SAND	NP	NP	NIL	62	38	NIL	2.57	SM
B404 BH1	9.70 - 10.20	Non plastic sandy SILT	NP	NP	3	43	54	NIL	2.38	MH
B404 BH1	15.00 - 15.50	Low plastic clayey SILT	48	11	NIL	8	92	NIL	-	MH
B404 BH2	5.50 - 6.00	High plastic clayey SILT	59	18	NIL	6	94	NIL	2.66	MH
B404 BH2	9.00 - 9.50	High plastic sandy SILT	56	16	1	32	67	NIL	2.49	MH
B404 BH2	15.00 - 15.50	Non plastic sandy SILT	NP	NP	2	38	60	NIL	-	-
B205 - BH1	7.50 - 8.00	Low plastic sandy SILT	32	13	NIL	40	60	-	2.58	ML
B205 - BH1	10.50 - 11.00	Non plastic sandy SILT	NP	NP	NIL	25	75	NIL	-	-
B205 - BH2	9.30 - 9.80	High plastic clayey SILT	51	17	NIL	6	94	30	2.5	MH
B205 - BH2	13.50 - 14.00	Low plastic sandy SILT	44	13	NIL	33	67	-	-	ML
B206 - BH1	6.50 -7.00	High plastic silty CLAY	76	42	NIL	6	94	100	2.59	CH
B206 - BH1	10.00 - 10.50	Low plastic sandy SILT	42	16	NIL	24	76	-	-	ML
B206 - BH1	14.00 - 14.50	Low plastic sandy SILT	37	12	NIL	25	75	-	-	ML
B206 - BH2	4.50 - 500	Low plastic sandy SILT	NP	NP	NIL	22	78	-	2.47	-
B206 - BH2	8.00 - 8.50	High plastic	52	18	NIL	15	85	-	-	MH

		sandy SILT								
B206 - BH2	12.50 - 13.00	Low plastic sandy SILT	39	16	NIL	35	65	-	-	ML
B207 - BH1	6.00 - 6.50	Non plastic silty SAND	NP	NP	NIL	53	47	-	-	-
B207 - BH1	11.20 - 11.70	Low plastic sandy SILT	39	15	NIL	21	79	2.55	-	ML
B207 - BH1	16.00 - 16.50	Low plastic sandy SILT	51	21	NIL	19	81	-	-	ML
B207 - BH2	5.50 - 6.00	High plastic silty CLAY	76	44	NIL	11	89	90	2.65	CH
B207 - BH2	9.65 - 10.65	High plastic clayey SILT with sand	49	4	NIL	14	86	-	2.52	ML
B207 - BH2	14.15 - 14.65	Low plastic sandy SILT	41	18	NIL	17	83	-	-	ML
B208 - BH1	9.00 - 9.50	Low plastic sandy SILT	41	18	NIL	17	83	-	2.61	ML
B208 - BH1	12.50 - 13.00	High plastic clayey SILT	55	23	NIL	2	98	-	-	MH
B208 - BH1	16.00 - 16.50	Low plastic sandy SILT	NP	NP	NIL	40	60	-	-	-
B208 - BH2	5.50 - 6.00	High plastic clayey SILT	63	33	NIL	11	89	70	-	MH
B208 - BH2	9.00 - 9.50	High plastic clayey SILT	84	34	NIL	3	97	80	2.53	MH
B208 - BH2	13.00 - 13.50	High plastic clayey SILT	63	33	NIL	3	97	70	-	MH
B209 - BH1	7.50 - 8.00	High plastic clayey SILT	69	27	NIL	4	96	-	-	MH
B209 - BH1	12.00 - 12.50	Non plastic silty SAND	NP	NP	NIL	54	46	-	-	-

B209 - BH2	12.00 - 12.50	Low plastic silty SAND	34	17	NIL	63	37	-	-	SM
B210 - BH1	6.50 - 7.00	Low plastic sandy SILT	43	19	NIL	15	85	-	-	MH
B210 - BH1	11.50 - 12.00	Low plastic sandy SILT	42	19	NIL	21	79	-	-	ML
B210 - BH1	15.00 - 15.50	Low plastic sandy SILT with gravel	49	21	15	16	69	-	-	ML
B210 - BH2	6.00 - 6.50	Non plastic sandy SILT	NP	NP	NIL	28	72	-	-	-
B210 - BH2	11.50 - 12.00	Non plastic sandy SILT	NP	NP	NIL	36	64	-	-	-
B211 - BH1	14.50 - 15.00	High plastic clayey SILT	53	20	NIL	3	97	50	-	MH
B211 - BH2	12.50 - 13.00	Low plastic sandy SILT	38	14	NIL	43	57	-	-	ML
B211 - BH1	5.50 - 6.00	High plastic clayey SILT	99	54	NIL	1	99	100	-	MH
B211 - BH1	9.00 - 9.50	High plastic clayey SILT	55	24	NIL	6	94	-	2.59	MH
B212 - BH1	5.00 - 5.50	High plastic clayey SILT	107	53	NIL	1	99	100	-	MH
B212 - BH1	7.00 - 7.50	High plastic clayey SLITwith sand	81	31	NIL	14	86	80	2.52	MH
B212 - BH1	13.00 - 13.50	Low plastic clayey SILT	45	19	NIL	6	94	NIL	-	ML
B212 - BH2	5.00 - 5.50	High plastic clayey SILT	102	51	NIL	1	99	90	-	MH
B212 - BH2	13.00 - 13.50	High plastic clayey SILT	67	32	NIL	4	96	50	-	MH
B212 - BH2	7.00 - 7.50	High plastic clayey SILT with sand	101	48	NIL	14	86	30	-	MH

B213 - BH1	9.50 - 10.00	Non plastic silty SAND	NP	NP	9	56	45	-	-	-
B213 - BH2	6.50 - 7.00	High plastic clayey SILT	93	43	NIL	8	98	-	-	MH
B213 - BH2	10.50 - 11.00	Non plastic silty SAND with gravel	NP	NP	25	44	31	-	-	-
B213 - BH2	13.50 - 14.00	High plastic clayey SILT	107	53	NIL	25	75	10	-	MH
B213 - BH2	14.50 - 15.00	Low plastic sandy SILT	49	20	NIL	34	66	NIL	-	ML
B214 - BH1	4.50 - 5.00	High plastic clayey SILT	103	52	NIL	2	98	70	2.71	MH
B214 - BH1	8.50 - 9.00	High plastic clayey SILT	103	52	NIL	2	98	100	-	MH
B214 - BH1	12.50 - 13.00	Low plastic clayey SILT	41	15	NIL	6	94	NIL	-	ML
B214 - BH2	4.50 - 5.00	High plastic clayey SILT	102	50	NIL	6	99	-	-	MH
B214 - BH2	9.45 - 9.95	High plastic sandy SILT	64	23	NIL	22	78	NIL	-	MH
B214 - BH2	12.30 - 12.80	High plastic clayey SILT	56	22	NIL	7	93	60	-	MH
B315 - BH1	6.50 - 7.00	Non plastic clayey SILT	NP	NP	NIL	1	99	-	-	-
B315 - BH1	9.50 - 10.00	Low plastic sandy SILT with gravel	NP	NP	21	25	54	-	-	-
B315 - BH1	14.50 - 15.00	High plastic clayey SILT	63	25	NIL	6	94	-	-	MH
B315 - BH2	6.50 - 7.00	High plastic clayey SILT	104	59	NIL	2	98	100	-	MH
B315 - BH2	16.30 - 16.80	High plastic sandy SILT	54	21	8	16	76			MH

B316 - BH1	12.30 - 12.80	High plastic clayey SILT	76	34	NIL	10	90	100	-	MH
B316 - BH2	5.00 - 5.50	High plastic clayey SILT	97	47	NIL	1	100	90	-	MH
B316 - BH2	13.00 - 13.50	Low plastic silty SAND	36	18	1	53	46	60	-	ML
B316 - BH2	9.50 - 10.00	Low plastic sandy SILT	48	20	NIL	34	66	40	-	ML
B317 - BH1	8.50 - 9.00	High plastic clayey SILT	88	41	4	1	95	-	-	MH
B317 - BH1	14.50 - 15.00	High plastic clayey SILT	94	52	NIL	2	98	-	-	MH
B318 - BH1	5.00 - 5.50	High plastic clayey SILT	87	44	1	2	97	-	2.77	MH
B318 - BH1	9.00 - 9.50	High plastic silty CLAY	101	62	NIL	2	98	-	-	CH
B318 - BH1	15.00 - 15.50	High plastic sandy SILT	66	32	9	14	79	-	-	MH
B318 - BH2	6.50 - 7.00	High plastic clayey SILT	93	48	NIL	1	99	-	-	MH
B319 - BH1	6.50 - 7.00	High plastic silty CLAY	89	50	1	3	96	-	2.5	CH
B319 - BH1	9.00 - 9.50	High plastic silty CLAY	96	56	NIL	1	99	-	-	CH
B319 - BH1	15.00 - 15.50	High plastic clayey SILT	48	14	NIL	4	96	-	-	ML
B319 - BH2	5.00 - 5.50	High plastic silty CLAY	90	51	1	6	93	-	2.59	CH
B319 - BH2	8.50 - 9.00	High plastic silty CLAY	102	59	NIL	1	H	-	-	CH
B319 - BH2	14.50 - 15.00	High plastic silty CLAY	53	22	NIL	7	93	-	-	CH
B320 - BH1	4.50 - 5.00	High plastic clayey SILT	93	45	NIL	1	99	-	2.42	MH

B320 - BH1	8.50- 9.00	High plastic silty CLAY	91	47	NIL	1	99	-	-	MH
B320 - BH1	14.50 - 15.00	High plastic silty CLAY	55	23	NIL	2	98	-	-	CH
B320 - BH2	3.80 - 4.20	High plastic clayey SILT	106	59	NIL	1	99	-	2.44	MH
B11-52 BH1	5.00 - 5.50	High plastic clayey SILT	53	24	NIL	6	94	50	-	MH
B11-52 BH1	5.50 - 6.00	High plastic silty CLAY	53	26	NIL	5	95	-	-	CH
B11-52 BH1	8.00 - 8.50	Low plastic sandy SILT	43	12	NIL	26	74	NIL	-	ML
B11-52 BH1	9.00 - 9.50	High plastic clayey SILT	55	13	NIL	6	94	-	2.35	MH
B11-52 BH1	12.50 - 13.00	Low plastic sandy SILT	36	15	NIL	25	75	30	-	ML
B11-52 BH2	3.50 - 4.00	High plastic clayey SILT with few sand	86	46	NIL	12	88	80	-	MH
B11-52 BH2	4.85 - 5.35	High plastic clayey SILT	73	37	NIL	3	97	60	-	MH
B11-52 BH2	6.75 - 7.25	High plastic clayey SILT	61	23	NIL	2	98	60	-	MH
B11-52 BH2	7.60 - 8.10	Non plastic sandy SILT	NP	NP	NIL	56	44	-	-	-
B11-52 BH2	9.80 - 10.30	Non plastic sandy SILT	NP	NP	NIL	36	64	NIL	-	-
B11-53 BH2	4.00 - 4.50	Non plastic sandy SILT	NP	NP	NIL	21	79	NIL	-	MH
B11-53 BH2	7.00 - 7.50	Low plastic sandy SILT	47	17	NIL	17	83	30	-	ML
B11-53 BH2	10.00 - 10.50	Low plastic sandy SILT	46	15	NIL	16	84	20	-	ML

B11-54 BH1	4.80 - 5.30	High plastic clayey SILT	97	49	NIL	2	98	110	-	MH
B11-54 BH2	3.60 - 4.10	High plastic clayey SILT	99	42	NIL	1	99	100	2.65	MH
B11-54 BH2	5.00 - 5.50	High plastic clayey SILT	75	41	NIL	2	98	-	-	MH
B11-54 BH2	9.40 - 9.90	Low plastic sandy SILT	43	11	NIL	19	81	NIL	2.64	ML
B11-55 BH2	4.90 - 5.40	High plastic clayey SILT	104	51	NIL	1	100	100	2.7	MH
B11-56 BH1	5.00 - 5.50	High plastic clayey SILT	102	49	NIL	4	96	100	2.56	MH
B11-56 BH1	7.50 - 8.00	High plastic clayey SILT	79	31	NIL	18	82	-	-	MH
B11-56 BH2	7.00 - 7.50	Non plastic sandy SILT	NP	NP	NIL	25	75	NIL	-	-
B11-56 BH2	11.00 - 11.50	Low plastic clayey SILT with few sand	50	15	NIL	12	88	-	-	ML
B11-56 BH2	11.50 - 12.00	Non plastic silty SAND	NP	NP	NIL	56	44	-	-	-
B11-57 BH1	4.50 - 5.00	Non plastic sandy SILT	NP	NP	NIL	29	71	-	-	-
B11-57 BH1	7.00 - 7.50	Non plastic sandy SILT	NP	NP	NIL	21	79	-	-	-
B11-57 BH1	11.00 - 11.50	Non plastic sandy SILT	NP	NP	2	33	65	-	-	-
B11-57 BH2	3.50 - 4.00	High plastic sandy SILT	67	30	NIL	17	73	-	-	MH
B11-57 BH2	7.50 - 8.00	Non plastic sandy SILT	NP	NP	2	43	55	-	-	-
B11-57 BH2	12.00 - 12.50	Non plastic sandy SILT	NP	NP	NIL	37	63	-	-	-

B11-58 BH1	5.50 - 6.00	Low plastic sandy SILT	48	15	NIL	31	69	-	-	ML
B11-58 BH2	5.00 - 5.50	High plastic clayey SILT	67	22	NIL	11	89	-	-	MH
B11-58 BH2	7.60 - 8.10	High plastic clayey SILT with sand	63	24	NIL	14	86	-	-	MH

Results of index test on disturbed soil

on parcel 6,8,9,11										
BH ID	Depth, m	Lab. Description	Atterberg limit	% Gravel	% Sand	% Silt	GS	FS %	Group symbol	
			LL	PI						
B4-21 - BH1	5.00 - 5.50	High plastic clayey SILT	96	44	MH	80	-	NIL	1	99
B4-21 - BH1	7.50 - 8.00	High plastic clayey SILT	93	44	MH	40	2.7	9	3	88
B4-21 - BH1	11.00 - 11.50	High plastic clayey SILT	101	47	MH	110	-	NIL	1	99
	4.50 - 5.00	High plastic clayey SILT	107	59	MH	140	2.58	NIL	2	98
B4-21 BH2	7.00 - 7.50	High plastic clayey SILT	90	43	MH	80	2.58	NIL	2	98
	3.50 - 4.00	High plastic clayey SILT	101	47	MH	100	2.67	9	3	88
	7.50 - 8.00	High plastic clayey SILT	100	48	MH	90	2.55	NIL	0	100
B4-22 BH1	12.50 -13.00	Low plastic sandy SILT	47	19	MH	10	-	NIL	35	65
	4.50 - 5.00	High plastic clayey SILT	95	50	MH	100	2.66	NIL	8	92

B4-22 BH2	10.50 -11.00	Low plastic sandy SILT	42	16	ML	20	2.53	NIL	23	78
B4-22 - BH2	7.00 - 7.50	High plastic clayey SILT	105	52	MH	100	-	NIL	8	99
B10-45 BH-1	6.15 - 6.75	Low plastic sandy SILT	48	16	ML	NIL	2.61	NIL	26	74
B10-45 BH1	10.00 - 10.50	Non plastic silty SAND	NP	NP	SM	NIL	-	1	50	49
B10-45 BH2	7.05 - 7.55	High plastic clayey SILT	64	27	MH	10	2.56	NIL	4	96
B10-45 BH2	10.50- 11.00	Low plastic sandy SILT	47	12	MH	NIL	-	NIL	22	78
B10-46 BH1	4.50 - 5.00	Non plastic silty GTRAVEL with sand	NP	NP	GM	NIL	-	54	21	25
B10-46 BH1	13.40 - 13.90	Low plastic clayey SILT with sa	46	10	ML	NIL	-	1	13	86
B10-46 BH2	4.50 - 5.00	Non plastic sandy SILT	NP	NP	-	NIL	-	NIL	25	75
B10-46 BH2	7.50 - 8.00	Low plastic sandy SILT	40	23	CL	10	2.6	1	29	70
B10-46 BH2	12.00 - 12.50	High plastic sandy SILT	54	15	MH	50	-	NIL	19	81
B10-47 BH1	4.50 - 5.00	High plastic clayey SILT	103	60	MH	-	2.48	NIL	1	99
B10-47 BH1	8.50 - 9.00	Non plastic sandy SILT	NP	NP	-	-	2.7	2	34	64
B10-47 BH1	11.0 - 11.50	Non plastic silty GRAVEL with sand	NP	NP	-	-	-	45	19	36
B10-47 BH2	4.00 - 4.50	Non plastic sandy SILT	NP	NP	-	2.62	-	NIL	23	77
B10-47 BH2	7.50 - 8.00	Non plastic sandy SILT	37	18	ML	-	-	NIL	31	69
B10-47 BH2	12.50 - 13.00	High plastic	NP	NP	MH	-	-	NIL	20	80

		sandy SILT								
B10-48 BH1	4.00 - 4.50	Non plastic gravelly SILT with sand	NP	NP	-	NIL	2.87	16	15	69
B10-48 BH1	4.50 - 5.00	High plastic clayey SILT	95	50	MH	60	-	NIL	1	99
B10-48 BH1	7.00 - 7.50	Low plastic sandy SILT	46	25	ML	20	2.59	NIL	18	82
B10-48 BH1	7.50 - 8.00	High plastic sandy SILT	NP	NP	-	NIL	-	NIL	35	65
B10-48 BH1	10.50 - 11.00	Non plastic SAND silty	NP	NP	SM	NIL	-	NIL	66	34
B10-48 BH2	6.50 - 7.00	High plastic clayey SILT	99	42	MH	80	2.72	NIL	0.4	99.6
B10-48 BH2	10.45 - 10.95	Non plastic sandy SILT	NP	NP	-	NIL	2.85	8	37	55
B10-49 BH1	4.90 - 5.50	High plastic clayey SILT	97	46	MH	100	2.7	NIL	2	98
B10-49 BH1	7.35 - 7.85	Non plastic silty with SAND	NP	NP	-	NIL	2.44	4	20	76
B10-49 BH1	11.50 - 12.00	Non plastic sandy SILT	NP	NP	-	NIL		NIL	26	74
B10-49 BH2	5.00 - 5.50	High plastic clayey SILT	100	51	MH	100	2.87	NIL	2	98
B10-49 BH2	8.80 - 9.30	Low plastic sandy CLAY	46	20	CL	NIL	2.57	NIL	21	79
B10-49 BH2	12.80 - 13.30	High plastic sandy SILT	61	24	MH	30	-	3	16	81
B10-50 BH1	7.00 - 7.50	High plastic clayey SILT with sandy SILT	60	25	MH	30	-	NIL	13	87
B10-50 BH2	4.50 - 5.00	Non plastic sandy SILT	NP	NP	-	NIL	2.66	NIL	17	83
B10-50 BH2	7.50 - 8.00	High plastic clayey SILT	57	22	MH	40	2.55	NIL	4	96
B10-50 BH2	10.50 - 11.00	High plastic clayey SILT	59	29	CH	70	-	NIL	3	97
B10-51 BH1	5.00 - 5.50	High plastic clayey SILT	68	33	MH	60	2.7	NIL	5	95
B10-51 BH1	8.00 - 8.50	LOW plastic clayey SILT	40	14	ML	20	-	NIL	15	85

B10-51 BH1	11.00 - 11.50	Non plastic silty SAND with gravel	NP	NP	-	NIL	-	29	31	40
B10-51 BH2	3.50 - 4.00	Non plastic silty GRAVEL with sand	NP	NP	-	NIL	2.58	30	29	41
B10-51 BH2	6.45 - 6.95	High plastic clayey SILT	63	25	MH	60	-	NIL	5	95
B10-51 BH2	11.00 - 11.50	High plastic clayey SILT with few sand	51	14	MH	NIL	-	NIL	13	87
B9-39 BH1	4.50 - 5.00	High plastic clayey SILT	100	44	MH	130	2.55	NIL	3	97
B9-39 BH2	5.00 - 5.50	High plastic silty CLAY	99	59	CH	110	-	NIL	2	98
B9-39 BH2	7.70 - 8.20	Non plastic sandy SILT	NP	NP	NIL	-	-	6	34	60
B9-40 BH1	4.50 - 5.00	High plastic silty CLAY	107	66	CH	-	2.81	NIL	1	99
B9-40 BH1	7.50 - 8.00	Non plastic sandy SILT	NP	NP	-	-	-	NIL	16	84
B9-40 BH2	4.50 - 5.00	High plastic clayey SILT	105	58	MH	-	2.83	NIL	1	99
B9-40 BH2	9.00 - 9.50	Non plastic silty GRAVE	NP	NP	-	-	-	65	10	25
B9-41 BH1	4.50 - 5.00	High plastic clayey SILT	99	48	MH	-	2.69	NIL	1	99
B9-41 BH1	8.50 - 9.00	High plastic clayey SILT	96	44	MH	90	-	NIL	2	98
B9-41 BH1	11.50 - 12.00	Non plastic silty SAND with gravel	NP	NP	-	-	-	17	34	49
B9-41 BH2	4.50 - 5.00	High plastic clayey SILT	97	54	MH	110	2.63	NIL	1	99
B9-41 BH2	8.50 -9.00	High plastic clayey SILT	99	55	MH	100	-	NIL	2	98
B9-42 BH1	13.50 - 14.00	Non plastic sandy SILT	NP	NP	-	-	-	3	41	56
B9-42 BH2	3.50 - 4.00	High plastic	116	62	MH	100	2.6	NIL	1	99

		clayey SILT								
B9-42 BH2	4.50 - 5.00	High plastic silty CLAY	113	73	CH	120	2.63	NIL	1	99
B9-42 - BH2	7.50 - 8.00	High plastic clayey SILT	98	46	MH	-	-	NIL	1	99
B9-42 - BH2	8.50 - 9.00	High plastic clayey SILT	106	54	MH	-	-	NIL	1	99
B9-42 - BH2	11.30 - 11.80	High plastic clayey SILT	106	53	MH	-	-	NIL	1	99
B9-43 - BH1	3.60 - 4.10	High plastic clayey SILT	99	50	MH	100	2.85	NIL	1	99
B9-43 - BH1	4.50 - 5.00	High plastic silty CLAY	101	54	CH	100	2.72	1	1	99
B9-43 - BH1	7.50 - 8.00	High plastic silty CLAY	103	66	CH	110	2.69	NIL	3	97
B9-43 - BH2	4.50 - 5.00	High plastic claye SILT	101	54	MH	90	-	1	1	98
B9-43 - BH2	7.50 - 8.00	High plastic clayey SILT	106	52	MH	100	-	NIL	1	99
B9-43 - BH2	10.00 - 11.00	Non plastic silty GRAVEL with sand	NP	NP	-	-	-	29	26	45
B9-44 BH1	7.50 - 8.00	High plastic clayey SILT	58	25	MH	80	-	NIL	10	90
B9-44 BH2	5.00 - 5.50	High plastic clayey SILT	108	57	MH	100	2.4	NIL	1	99
B9-44 BH2	8.30 - 8.80	Low plastic clayey SILT	48	17	ML	-	-	NIL	1	99
B8-36 BH1	5.05 - 5.55	Low plastic sandy SILT	45	15	ML	-	2.58	NIL	20	80
B8-36 BH2	6.50 - 7.00	Low plastic sandy SILT	43	15	ML	-	-	NIL	16	84
B8-37 BH1	3.80 - 4.30	Non plastic gravelly SILT with sand	NP	NP	-	-	-	28	18	54
B8-37 BH1	10.50 - 11.00	Non plastic gravelly SILT	42	16	ML	40	-	NIL	20	80

		with sand								
B8-38 BH1	4.00 - 4.50	Low plastic silty GRAVEL	50	12	GM	-	2.6	38	17	45
B8-38 BH1	10.50 - 11.00	High plastic sandy SILT	61	22	MH	50	-	NIL	16	84
B8-38 BH2	11.50 - 12.00	Low plastic silty GRAVEL with sand	48	15	GM	-	-	38	17	45

Results of index test on disturbed soil parcel 7,10,12

			Atterberg Limit		% Gravel	% Sand	% Fine	FS %	GS	Group Symbol
BH ID	Depth, m	Lab. Description	LL	PI						
B5-23 BH1	3.50 - 4.00	High plastic clayey SILT	69	24	NIL	1	99	-	-	MH
B5-23 BH1	9.00 - 9.50	High plastic silty CLAY	95	56	NIL	2	98	105	2.66	CH
B5-23 BH1	12.50 - 13.00	High plastic silty CLAY	104	62	NIL	1	99	-	-	CH
B5-23 BH2	4.50 - 5.00	High plastic silty CLAY	104	66	NIL	2	98	110	2.63	CH
B5-23 BH2	7.50 - 8.00	High plastic silty CLAY	101	63	1	1	98	-	-	CH
B5-23 BH2	13.00 - 13.50	High plastic silty CLAY	102	67	NIL	2	98	-	-	CH
B5-24 BH1	7.40 - 7.90	High plastic silty CLAY	103	62	NIL	2	98	100	2.6	CH
B5-24 BH1	11.00 - 11.50	High plastic sandy CLAY	130	20	1	21	78	-	-	MH
B5-24 BH1	14.00 - 15.00	High plastic clayey SILT	38	31	NIL	13	87	-	-	MH
B5-24 BH2	6.500 - 7.00	High plastic silty CLAY	97	60	NIL	2	98	100	2.67	MH

B5-24 BH2	11.50 - 12.00	Non plastic sandy SILT	NP	NP	NIL	28	72	-	-	-
B5-24 BH2	15.00 - 15.0	High plastic sandy SILT	68	22	6	15	79	-	-	MH
B5-25 BH1	6.50 - 7.00	High plastic clayey SILT	96	48	NIL	1	99	50	2.84	MH
B5-25 BH1	14.00 - 14.50	High plastic clayey SILT	72	25	1	7	92	NIL	-	MH
B5-25 BH2	5.50 - 6.00	High plastic silty CLAY	99	60	NIL	1	99	-	-	CH
B5-25 BH2	8.50 - 9.00	SILT High plastic clayey	103	49	NIL	1	99	30	-	MH
B5-25 BH2	9.50 - 10.00	High plastic clayey SILT	94	45	NIL	2	98	30	-	MH
B5-25 BH2	14.50 - 15.00	Low plastic sandy SILT	43	15	1	21	78	20	-	ML
B5-26 BH1	5.50 - 6.00	High plastic clayey SILT	97	52	NIL	1	99	80	2.71	MH
B5-26 BH1	10.55 - 11.05	Low plastic sandy SILT	39	16	NIL	3	97	60	-	ML
B5-26 BH2	6.50 - 7.00	High plastic silty CLAY	102	60	NIL	1	99	60	2.69	CH
B5-26 BH2	10.00 - 10.50	Low plastic silty GRAVEL/SAND	NP	NP	26	26	48	NIL	-	-
B5-27 BH1	4.50 - 5.00	High plastic clayey SILT	93	42	NIL	1	99	20	2.77	MH
B5-27 BH1	8.55 - 9.05	Low plastic sandy CLAY	34	15	NIL	23	77	30	-	CL
B5-27 BH2	5.00 - 5.50	High plastic clayey SILT	115	57	NIL	1	99	100	2.32	MH
B5-27 BH2	8.50 - 9.00	High plastic clayey SILT	103	49	NIL	1	99	-	-	MH
B5-27 BH2	14.50 - 15.00	High plastic sandy SILT	43	15	1	21	78	-	-	ML

B6-28 BH1	6.50 - 7.00	High plastic clayey SILT	98	40	NIL	1	99	80	-	MH
B6-28 BH1	9.00 - 9.50	Low plastic sandy SILT	33	10	NIL	30	70	NIL	-	ML
B6-28 BH1	12.00 - 12.50	Low plastic clayey GRAVEL with sand	33	10	58	18	24	NIL	-	GC
B6-29 BH1	8.00 - 8.50	High plastic gravelly SILT with sand	46	16	NIL	16	84	NIL	-	ML
B6-29 BH1	11.00 - 11.50	Low plastic clayey GRAVEL with sand	38	14	56	20	24	NIL	-	GC
B6-29 BH2	6.00 - 6.50	High plastic clayey SILT	106	44	NIL	1	99	100	-	MH
B6-29 BH2	8.50 - 9.00	Low plastic sandy SILT	34	8	NIL	30	70	NIL	-	ML
B6-29 BH2	12.00 - 12.50	High plastic clayey SILT	71	24	NIL	3	97	80	-	MH
B6-30 BH1	5.50 - 6.00	High plastic clayey SILT	99	52	NIL	2	98	100	-	MH
B6-30 BH1	6.50 - 7.00	High plastic clayey SILT	88	45	NIL	3	97	80	-	MH
B6-30 BH1	9.50 - 10.00	Non plastic silty GRAVEL with sand	NP	NP	60	24	16	NIL	-	GM
B6-30 BH1	10.50 - 11.00	Non plastic silty GRAVEL with sand	NP	NP	32	30	38	NIL	-	GM
B7-32 BH1	4.50 - 5.00	High plastic sandy SILT	80	31	NIL	15	85	-	-	MH
B7-32 BH1	8.00 - 8.50	High plastic sandy SILT	61	12	NIL	8	92	-	-	MH
B7-32 BH2	5.00 - 5.50	High plastic sandy SILT	61	29	NIL	4	96	-	-	MH
B7-33 BH1	6.50 - 7.00	High plastic clayey SILT	102	53	NIL	1	99	-	-	MH

B7-33 BH1	9.50 - 10.00	High plastic clayey SILT	56	24	NIL	10	90	-	-	MH
B7-33 BH2	4.50 - 5.00	High plastic clayey SILT	100	47	NIL	1	99	-	-	MH
B7-33 BH2	7.00 - 7.50	High plastic sandy SILT	65	32	NIL	14	86	-	-	MH
B7-34 BH1	5.50 - 6.00	High plastic clayey SILT	112	66	NIL	1	99	105	-	MH
B7-34 BH1	9.00 - 9.50	High plastic clayey SILT	50	19	NIL	10	90	-	2.32	MH
B7-34 BH1	15.00 - 15.50	Non plastic sandy SILT	NP	NP	NIL	16	84	-	-	-
B7-34 BH2	5.00 - 5.50	High plastic clayey SILT	103	48	NIL	1	99	-	-	MH
B7-34 BH2	5.50 - 6.00	High plastic silty CLAY with few sand	54	24	NIL	13	87	-	2.65	CH
B7-34 BH2	14.00 - 14.50	Non plastic sandy SILT	NP	NP	NIL	32	68	-	-	-
B7-35 BH1	9.50 - 10.00	High plastic clayey SILT with sand	57	21	1	13	86	-	-	MH
B7-35 BH1	14.20 - 14.70	Non plastic sandy SILT	NP	NP	1	48	51	-	-	-
B7-35 BH2	11.50 - 12.00	High plastic sandy SILT	54	17	NIL	15	85	-	-	MH

Appendix 2: In Beshale site protection of the excavated expansive soil from sliding

Appendix 3: In Beshale site sliding of excavated expansive soil between two blocks. Previously their two blacks' gap was 5m

Appendix 4: In order to excavated maximum 2 m from the boundary of the blocks. However; due to extensive dominance of expansive soil it extending to 8 m.

Certificate

This is to certify that the thesis prepared by **Mr. Tesfalem G/Mariam** entitled **“Consolidation Behavior of Expansive Soil: The Case of Bole Beshale in Addis Ababa”** and submitted in fulfillment of the requirements for the Degree of Master of Science complies with regulations of the University and meets the accepted standards with respect to originality and quality.

Signed by Examining Board:

Examiner: Dr. Aregaw Asha ________________ _____________

Signature Date

Examiner: Melaku Sisay ____________________ _____________

Signature Date

Thesis Advisor Dr. Trufat Hailemariam _______________ _____________

Signature Date